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Web crippling at seat angle supports, 1941

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WEB CRIPPLING AT SEAT ANGLE SUPPORTS

by

Bruce Johnston and Gerald G. Kubo

FOREWORD

The following report to the American Institute of Steel Construction covers several web crippling tests authorized by the A.I.S.C. Technical Research Committee in July 1940. The first of these tests was a preliminary test made by Robert Hechtman, studying the distribution of vertical web strain at the end of the beam for different locations of roller support. The main program of four tests was carried out by the authors in accordance with suggestions made by the Bethlehem Steel Company. The only variable in these four tests was the stiffness of the seat angle support, used in each case in conjunction with a flexible bolted top angle. Detailed observations were made as to strains, local displacements and general behavior. Another program furnished two tests of a welded seat angle and top plate detail which failed due to web crippling and buckling. The results of all of these tests are summarized and conclusions and design recommendations presented insofar as the limited number of tests makes possible.

INTRODUCTION

Various types of general behavior which may be expected locally at a beam support will be reviewed. In Fig. 1a is shown the seat angle support used in building construction. If the seat angle is flexible or out of square (as shown in Fig. 1d) the reaction resultant at low loads may be near the end of the beam. Initial yielding in the web of the beam will spread as shown in Fig. 1f and the resultant will move toward the center of the yielded portion.

After the yielded zone has spread over a sufficient area local plastic buckling or "crippling" may occur in the plastic region. This may be followed or accompanied by more general web buckling as shown in Fig. 1g, which takes place over a larger area than the initial plastic zone. "Web crippling" is predominantly a plastic behavior affecting a very local region and "web buckling" is predominantly an elastic behavior affecting a relatively larger part of the web. Both types of behavior are affected by several factors not considered as variables in this program. In ordinary building construction one of the factors is the participation of the detail at the top of the beam. If this detail is stiff enough to participate in carrying the load, as in Fig. 1h, it may inhibit general web buckling as well as carry part of the vertical end reaction. If the top detail is flexible such as the top plate in Fig. 1i, it will not participate to as great a degree in carrying the end reaction.

The very stiff seat angle or the stiffened beam seat is similar to the condition existing when a **sole** plate such as might be used in a foundation grillage or on a short span railroad bridge is used as shown in Fig. 1b. In this case the end rotation of the beam, if permitted, may cause the reaction resultant to move to the span side of the support as shown in Fig. 1e. Initial plastic yielding will cause the resultant to move back toward the end of the beam. Fig. 1c shows the laboratory setup frequently used in previous tests. With this setup the reaction resultant is definitely located and there is no external restraint to rotation of the lower beam flange at the support.

Empirical formulas for allowable end reaction have been of two types; (1) based on local web crushing or web crippling failure as in Fig. 1f, and (2) based on the possibility of buckling failure as shown in Fig. 1g. The web crushing type formula was presented in 1909 by Hudson[1] and in University of Illinois Bulletins[2,3] published in 1913 and 1916. This formula simply calculated the local compressive stress in the web on an average allowable basis as follows:

$$f = \frac{R}{bt}$$

f = compressive stress at root of flange fillet
R = reaction
b = bearing length
t = web thickness

The early Illinois Bulletins[2,3] also recognized the possibility of diagonal buckling of the web. A later Illinois Bulletin (No.241) recommended in 1932 a column type formula known as the Carnegie formula in preference to the Hudson Formula. This recommendation was based on tests of a large number of very light J & L beams. The following history of the development of these empirical formulas is abstracted from a memorandum prepared on December 26, 1940 by Mr. Jonathan Jones and Mr. C. H. Mercer of the Bethlehem Steel Company.

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1. Engineering News, December 9, 1909

2. H. F. Moore, "The Strength Of I-Beams In Flexure" University of Illinois Bulletin No. 68

3. H. F. Moore, and W. M. Wilson, "Strength Of Webs Of I-Beams And Girders" University of Illinois Bulletin No. 86

"In its first (1923) Specifications for Steel Buildings, the American Institute of Steel Construction derived a partially-analytical, partially-empirical formula, in which the Institute's Rankin-Gordon column formula

$$p = \frac{18,000}{1 + \frac{l^2}{18,000r^2}}$$

was applied to a column comprising the beam web, taken on a 1:2 sloping line to determine "l". This led to the relation

$$R = ft\ (B + \frac{d}{4}), \quad \text{where } f = \frac{18,000}{1 + \frac{d^2}{6000t^2}}$$

t = web thickness
d = overall depth of beam
R = allowable reaction
B = length of bearing

On this hypothesis, a column of the true web thickness, of a width arbitrarily taken as $(B + \frac{d}{4})$, and of the length of the 1:2 sloped line, fails by buckling at mid-depth.

"The Carnegie Pocket Companion of the same date followed the same method, but with a straight-line column formula

$$f = 19,000 - 173\ \frac{d}{t}$$

"By 1931 (the writer has none of the intermediate editions) the Pocket Companion had changed to the A.I.S.C. type formula, but with a quite different denominator, viz,

$$f = \frac{18,000}{1 + \frac{d^2}{3000t^2}}$$

"The Bethlehem Manuals of 1926 and 1931 show this same formula; a discussion in the 1934 Pocket Companion shows that the 6000 factor is derived by considering the beam held against twisting of the top flange, and the 3000 factor by considering it free. By 1934 the A.I.S.C. Carnegie and Bethlehem books agreed in giving tabular safe loads on the basis of top flange held laterally, which seems perfectly proper in view of standard building practice.

"It will be found that the derivation of the foregoing Rankine-type buckling formulas involves the conception that when d/t does not exceed 60, there will be no buckling and the full web shear allowance may be permitted.

"In the course of some experiments at Lehigh University, reported in Trans. A.S.C.E. Vol. 100 (1935) pp. 675-706, it was found that web buckling would not occur, because it would be preceded by another form of failure, if the ratio d/t was less than 70, possibly if less than 80. As this ratio is not as great as 70 for any American rolled beams, the web-buckling criterion and formula were removed as "excess baggage" from the 1936 A.I.S.C. Specification and tables of safe loads on beams. The Carnegie and Bethlehem handbooks having been discontinued in favor of the A.I.S.C. Manual, there is now no web-buckling rule in force for steel beams in buildings, except in such building codes as have not yet been brought up to date.

"In the same experiments which led to the elimination of web-buckling as a criterion, it was found that there was another possible criterion of failure, not yet recognized in specifications, in web crippling. This is a lateral crippling of the web just at the top of the flange fillets, over the support, where the entire reaction is obviously concentrated on a small area of web. This discovery, already anticipated by some individual engineers, led to the adoption in the 1936 A.I.S.C. Specification of the criterion or formula.

$$R = 24{,}000\ t\ (B + k)$$

where k is the depth from under side of bottom flange to top of fillet, i.e., the depth of the material giving lateral stability to the web itself.

"In the subsequent editions of the A.I.S.C. Manual, the tables contain this limitation on beam reactions. It should be borne in mind that this change in beam reactions was coincident with a general increase in beam loads by reason of increasing the tension allowance from 18,000 to 20,000, largely in recognition of the increase in specified yield point from 30,000 to 33,000.

"Prior to 1940, the A.R.E.A. Specifications for Railway Bridges did not have a web-buckling prescription. In that year one was adopted, viz.,

$$R = f\ t\ (B + \frac{d}{4})$$

where $$f = 15{,}000 - \frac{3}{4}\left(\frac{d}{t}\right)^2 \text{"}$$

The present A.I.S.C. formula is seen to be a modification of the old Hudson Formula.

TEST PROGRAM AND PROCEDURE

The specimens were detailed by Mr. Robert A. Hechtman in line with suggestions made by Mr. Jonathan Jones and Mr. C. H. Mercer of the Bethlehem Steel Company. Only four variations of one variable, namely, the seat angle stiffness, were introduced and only one specimen of each was tested. The beam in each test was a 12 WF 50 section 4 ft 11 in. long, supported between top and seat angle connections on column stubs. The setup for the tests is shown in Fig. 2. The unstiffened 6 by 4 by 8-1/2 in. long seat angles were 1/2, 3/4, and 1 in. thick in specimens ba, bc, and bb, respectively. The fourth connection used a 3/8-in. seat angle with two angle stiffeners for support. The top angle was a flexible 5/16-in. angle in each case. The connections were detailed to conform to standard shop practice.

All dimensions were carefully measured and it was found that the outside faces of the two flanges were not parallel in cross section. However, these variations were within allowable rolling tolerances of the A.I.S.C. In addition, the saw cutting of the beam ends and punching of holes had left ridges which were certain to introduce irregularities in the test behavior at low loads. In order to eliminate these uncertainties beams ba and bc were planed flat and square for a short distance in from the ends, and beams bb and bd were ground to a relatively level surface with a portable grinder. The amount of material taken off of the flanges was negligible. The outside faces of the seat angle legs were found in section to make an angle less than ninety degrees in all cases as shown in Fig. 1d, causing initial bearing to take place on the extreme end of the beam.

The details of the connections are given in the accompanying shop drawings, Fig. 3. The column posts were used twice, on opposite sides, and were of sufficient strength to suffer no appreciable permanent distortion under load. To prevent local bearing failure under the center load, a pair of stiffeners was welded on each side of the web. The beam and column stubs were lined up and bolted with as nearly as possible uniform tension in all of the bolts. Initial tension was put in the column tie backs (see Fig.2) to keep the columns from rotating appreciably during test.

Stresses at loads within the elastic range in the web just above the fillets over the seat angles were computed from strains measured by a battery of Huggenberger tensometers. A one-inch gage length was used. The location of the gages is shown in Fig. 4a and photographs of the Huggenbergers in position are shown in Fig. 4b and 4c. Strains were measured in both the vertical and horizontal direction, permitting the computation of direct stresses in the vertical direction.

The vertical deformation in the web and permanent set beyond the elastic range was measured by means of a 1/1000 Ames Dial deformeter gage especially built for these tests. The measurements were taken above the support, between the inside of the flange and an angle bolted to the web six inches above, as shown in Fig. 5a and 5b.

The lateral deformations of the web along a line 1/4 in. from the end of the beam were measured by the deformeter gage from a fixed reference plane. The readings were made at the locations shown in Fig. 6a and an illustration of the procedure is shown in Fig. 6b.

The general behavior of the connections throughout the tests was studied by means of vertical deflection readings of the outstanding toes of the top and seat angles. The dial arrangement for these measurements is shown in Fig. 7a, 7b, and 7c. The relative slip between the seat angle and column was also measured by means of Ames Dials, as shown in Fig. 7b. The arrangement for deflection and slip measurements on the stiffened seats required Ames dials on both sides of the stiffener as shown on Fig. 7c, whereas in the case of the unstiffened seats deflections were measured by single dials at the center.

TEST RESULTS

Physical Tests of Materials - Tension test coupons of beam web material were made with, across, and at 45° to the direction of rolling. Tension tests were also made of the seat and top angle material. The results of all tension tests are given in Table 1. Recent tests[4] have shown that the compression stress-strain characteristics of structural steel are practically the same as the tensile characteristics, hence the compression failure in the web of the beam may be correlated with the tensile stress-strain characteristics of the steel.

Stress Distribution in the Elastic Range - The distribution of average vertical direct stress in the web above the support was determined from the average vertical and horizontal strain readings measured over a one-inch gage length. The vertical stresses, σ_y, were determined by the stress-strain relationship for plane stress.

$$\sigma_y = \frac{E}{1-\nu^2}(\epsilon_y + \nu\epsilon_x)$$

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4. B. G. Johnston and Francis Opila
"Compression And Tension Tests Of Structural Alloys"
A. S. T. M. 1941 Meeting

The vertical compressive stresses above a concentrated reaction force diminish rapidly in the vertical direction. The maximum compression in the web at an end reaction occurs just at the tangent point of the web fillet at the end of the beam. The tensometers were placed along a horizontal line as close as possible to the web fillet but because of the one-inch gage length they necessarily represent an average stress which is only an approximation of the stress one-half inch above the line of web fillet tangency.

The vertical web stresses with the top angle unbolted are plotted for each of the four tests in Fig. 8. The shape of the curves for the 1/2 and 3/4-in. seat angles is quite similar to the theoretical stress distribution assuming a point load at the end of the beam. The vertical stress distribution based on the theory of the wedge[5] for plane stress is given for this special case by:

$$\sigma_y = - \frac{1.363R\ (1.571y^2 - yx)}{t\ (x^2 + y^2)^{3/2}} \qquad (1)$$

R = reaction at end of beam
y = vertical distance above seat angle
x = horizontal distance in from end of beam
t = web thickness

The compressive stress at the end of the beam where $x = 0$

$$\left[\sigma_y\right]_{x=0} = - \frac{2.141R}{ty} \qquad (2)$$

The preceding formulas are based on the theory of plane stress and therefore in the present application neglect the effect of the outstanding parts of the flange on the extension of the web. Assuming the theory to hold at the tangent of the fillet, where $y = k$, the maximum stress at the end of the beam would be

$$\left[\sigma_y\right]_{max} = - \frac{2.141R}{tk} \qquad (3)$$

It is of interest to compare Eq. 3 with the maximum stress at the fillet under a point force P applied to an interior section of the beam, in which case (Ref.5, p.83)

$$\left[\sigma_y\right]_{max} = - \frac{0.637R}{tk} \qquad (4)$$

A comparison of Eq. 3 and 4 indicates that a local concentration of reaction at the end of a beam such as shown in Fig. 1d, will cause maximum localized web stresses more than three times as great as a local concentration of reaction at an interior support such as in Fig. 1c.

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5. Timoshenko, "Theory of Elasticity" p. 93

Referring again to Fig. 8, it may be concluded that one or both of two factors caused relatively high concentration of stress at the end of the beam.

(1) The outstanding legs of the seat angles were out of square enough to result in contact only at the end of the beam. This is indicated particularly in the case of the 1/2 in. and 3/4-in. thick seat angles as shown in Fig. 8.

(2) The outstanding legs of the seat angles were not rigid enough to distribute the end reaction over the length of the outstanding leg.

The more uniform stress distribution shown in Fig. 8 for the one-inch seat angle and the stiffened seat indicates that these supports were stiff enough to distribute the end reaction load. The maximum measured stresses in this case were much less in the elastic range and the general shape of the stress distribution was roughly triangular. The foregoing observations as to stress distribution may be only of academic interest, since a practical design formula will probably be based on "limit design" allowing a partial yielding of the web and a consequent redistribution of stress.

Fig. 9 shows the web stress distribution for the same supports and same applied load as in Fig, 8 but with the top angles bolted. The shape of all of these curves is roughly triangular and it is evident that the 5/16-in. top angle participated appreciably in carrying the end reaction. The areas under the stress diagrams multiplied by the thickness of the web should equal the total end reaction carried by the seat angle and a comparison of these values is tabulated below: (units are in kips).

	Seat Angle 6x4x1/2	Seat Angle 6x4x3/4	Seat Angle 6x4x1	Seat Angle 6 x 4 x 3/8 Stiffener Angle 4 x 3-1/2 x 5/16
Top Angle Loose	13.7*	14.8	13.5	16.0
Top Angle Tight	11.2*	11.0	8.1	10.7
Carried by Top Angle	2.5	3.8	5.4	5.3
Percentage Carried by Top Angle	18.3	25.6	40.0	33.1

* No horizontal strain readings, values given are $E\epsilon$.

The foregoing computed reactions vary from the actual applied reaction of fifteen kips by as much as ten per cent. The discrepancy may be due in part to the fact that some of the reaction is carried by shear in the part of the web below the gages. However, the results consistently show an appreciable participation of the bolted top angle in carrying the load. The participation of the top angles was considerably more than would be expected on the basis of equal top and seat angle deflections, due probably to the fact that deformation of the beam web results in greater deflection in the top angle than in the seat angle.

Measurements in the Plastic Range - In order to develop practical values for working loads it is almost inevitable that the web of a beam above a support be stressed locally beyond the yield point. Such local overstrain may not be dangerous if the beam web is stable against buckling. Under stable conditions the yielding of the web above a support gradually spreads as more and more load is applied, and it is frequently very difficult to determine a usable "limit of structural usefulness". All of the tests beyond the elastic range were made with the top angles bolted tight.

The methods which were used to measure vertical and lateral deformations of the beam web were described on page 6. The diagrams in Fig. 10 show the test results of vertical deformation in the beam webs above the support plotted at various loads for each of the eight beam ends. Permanent sets based on the same data are shown in Fig. 11. These diagrams illustrate to some extent how the zone of plastic yielding spreads from the initially overstressed regions and thus gradually equalizes the stress distribution in the beam web above the aupport.

The spreading out of the plastic area is illustrated very well by the series of pictures taken at successive increments of load and shown in Fig. 12. Each picture is a similar view of the east end of specimen bd, above the stiffened seat. The area covered by the dark lines where the mill scale has flaked away is accentuated by whitewashing the beam web surface. This dark area shows approximately the yielded region in the beam web.

The horizontal extent of yielding in the beam web was measured at various load increments and in Fig. 13 these measurements are plotted against reaction load. In the upper range these curves all approach rather close to a straight line which is the equation of $R = 18.3l$, where l is the length of yield. An average stress in the yielded zone on the basis of this straight line is $\frac{18.3}{0.40} = 46$ kips per sq in., whereas the actual yield-point stress was about 40 kips per sq in.

This difference is probably due to the participation of the top angle in carrying the reaction. In the case of these tests and on the basis of these figures it might be inferred that an average working stress over the yielded portion of $\frac{46}{40} \times 20 = 23$ kips per sq in. could very well be permitted when top angles are as effective as in the present case. It is further noted by Fig. 13 that all of the supports except the 1/2-in. seat angle indicated that yielding without appreciable crippling would be developed at a maximum load of 87.5 kips over the full A.I.S.C. allowable bearing distance of $B + k = 3.5 + 1.25 = 4.75$ in.

The lateral deformations of the beam web along a vertical line 1/4-in. from the end of the beam are shown for various loads in Fig. 14. The plastic phenomenon of web crippling is illustrated very well by the behavior shown for specimen ba, where accelerated lateral deformation of the yielded portion of the beam web is particularly noticeable between the 80 and 85-kip loads. Plastic web crippling should be differentiated from the elastic buckling phenomenon which might occur in beams with very high depth-thickness ratios.

The curves of vertical deflections of the seat angle and top angle plotted against reaction load provide a graphical record of general connection behavior as shown in Fig. 15 and 16. The deflection of the top angle is structurally the most significant because it includes both the seat angle deflection and the compression in the beam web. The top angle deflection readings also would indicate to what extent settlement of a floor slab adjacent to a column might be possible. The deflections of the seat angle indicate the degree to which failure took place in this particular element. Fig. 15 shows that failure was most pronounced in the case of the 1/2-in. seat angle which had a deflection approximately four times that of the stiffest seat angle tested.

The difference between the curves in Fig. 15 and 16 for the east and west ends is at least in part due to the slight difference in set-back of the beam from the face of the column at the two ends.

The general behavior of all the connections may be compared on the basis of an arbitrary offset to the top angle load-deflection curves. In Fig. 16 an offset line is drawn parallel to the average of the three nearest reload curves. The intersection of this line with the curves of test results would indicate loads at which a permanent vertical set near the toe of the top angle at the end of the beam would be approximately 0.04 in. In cases of integral floor and wall construction such a displacement if transmitted to the adjacent material would result in visible cracks and on this basis these loads might indicate an upper limit of structural usefulness. However, the selection of 0.04-in. set as a basis for comparison was made on a purely arbitrary basis.

The condition of specimen bd after test as shown in Fig. 17 is typical of all of the specimens after test.

DESIGN METHODS

The four tests which were carried out by the authors are not sufficient in number and scope to form a basis for any general design procedure. The tests do provide detailed information regarding the type and progress of seat angle and beam web failure which may be expected in some typical designs similar to those tested.

An economical design method will be in line with the following reasoning. Since it is impossible in actual practice to avoid local concentrations of stress in the beam web it must be expected that the beam web will be locally stressed beyond the yield point at working loads. The redistribution of stress in the beam web will not be harmful provided web crippling or web buckling does not ensue. It is essential that the supporting seat angle or stiffened seat be strong enough and stiff enough to effect the necessary redistribution of stress in the beam web without itself deforming too much. Hence, either the beam web or the seat angle must be designed on a conservative, relatively non-yielding basis. It is obviously most economical to make the seat angle the object of the conservative design.

The foregoing is essentially the procedure proposed by the American Bridge Company design sheets which were recently used by the A.I.S.C. manual committee to arrive at allowable reaction values for seat angle supports. Copies of two A. B. Company sheets are given on the following page to provide a ready reference.

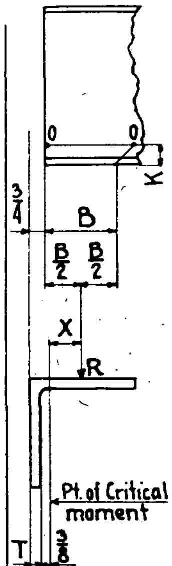
The notes provided for these design sheets are self-explanatory and the general method is based on the same line of reasoning as that adopted by one of the authors in a previous report on welded seat angles[6]. It is assumed that the length of bearing on the seat angle is limited to length "B" necessary to carry the reaction load at 24 kips per sq in. over a distance B + k.

The design reaction values proposed in the table depend on the web thickness, beam set-back as ordered, and the thickness of seat angle. It is evident that an assumption must be made as to "k" in terms of beam-web thickness and the result gives a reaction value for balanced design as to beam-web bearing and seat angle design. The proposed allowable stress is 24 kips per sq in. in both the beam-web and

6. "Designing Welded Frames", p.360s, The Welding Journal, October 1939, Research Supplement.

BEAM SEATS WITHOUT STIFFENERS

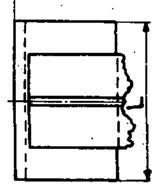
8.13a



① Beam ordered ½" from back of Seat with a possible underrun of ¼".

② R = Reaction of Beam.
$B = \frac{R}{Q_1} - K$, but not less than ½ (0-0).
Q_1 = Allowable pressure per linear inch along 0-0 for crushing, (24,000 lb. per sq. in.).
See A.I.S.C. Specifications 1939 Sec. 19h.
K = Distance from bottom of Beam to top of Fillet.

③ Thickness of Seat Angle figured for a Bending Moment RX on outstanding leg of Seat ⅜" from face of vertical leg of Seat with a fiber stress not over 24,000 lb. per sq. in. Fillets of Seat Angles are generally ½".
The 24,000 lb. unit is used, as the Seat Angle being fastened to Beam the outstanding leg is restrained and the true moment is somewhat less than for a simple Cantilever.
For a different fiber stress for bending on Seat, the value of R will be in proportion to the fiber stress used. Thus for a fiber stress of 18,000 lb. per sq. in. a Seat would be good for 18/24 (¾) of its value for 24,000 lb. per sq. in.



④ See Chart on Page 8.13b.
The values of R for a fixed B are in proportion to the length of Seat. The Chart is figured for Seats one inch long.
For a given R, B is first figured as per ②.
Then with a given length Angle for seat, find R/L (Reaction per linear inch of Seat).
Enter Chart on left for R/L and follow horizontal line until it intersects vertical line passing through figured B. The nearest curve above this intersection gives the thickness of Seat Angle required.

$$X = \frac{B}{2} + \frac{3}{4} - T - \frac{3}{8}$$

$$M = R(X) = 24{,}000\frac{LT^2}{6}$$

$$R = \frac{24{,}000\,LT^2}{6X} = \frac{4000\,LT^2}{X}$$

$$R(\text{per linear inch of Seat}) = \frac{4000\,T^2}{X}$$

L = Length of Seat Angle (max. effective length = 9").
T = Thickness of Seat Angle.
Curves on page 8.13b plotted from these equations.

VALUE OF SEAT ANGLES IN KIPS WITHOUT STIFFENERS

8.13

		Length: 6" O.S.L: 4						Length: 8" O.S.L: 4					
		Thickness of Seat L						Thickness of Seat L					
Web Thickness	Set Back	3/8	1/2	5/8	3/4	7/8	1	3/8	1/2	5/8	3/4	7/8	1
3/16	1/2	6	9	11	14	16		7	10	13	15	16	
	3/4	5	7	10	12	15	16	6	9	11	14	16	
	1		6	9	11	14	16	5	7	10	13	16	
1/4	1/2	8	11	14	17	20	23	9	12	16	19	22	23
	3/4	6	9	12	15	18	21	7	11	14	17	20	23
	1		8	10	13	16	19	6	9	12	15	19	23
5/16	1/2	10	15	18	21	25	28	11	16	20	24	27	31
	3/4	6	11	16	19	22	26	8	13	17	21	25	29
	1	5	8	13	17	20	23	6	10	15	19	22	26
3/8	1/2	11	17	22	26	30	34	12	19	24	28	32	35
	3/4	7	12	18	23	27	31	8	15	21	25	29	34
	1	5	9	14	20	23	27	6	11	17	22	26	31
7/16	1/2	12	18	25	30	34	35	13	21	27	32	35	
	3/4	7	13	20	26	31	35	9	15	23	28	33	35
	1	5	9	15	21	27	31	6	12	18	24	30	35
1/2	1/2	12	20	28	34	35		14	22	31	35		
	3/4	8	14	21	28	35		9	16	24	32	35	
	1	5	10	16	22	29	35	7	12	19	26	33	35
9/16	1/2	14	21	30	35			15	24	34	35		
	3/4	8	14	22	31	35		10	17	25	34	35	
	1	5	10	16	24	31	35	7	13	19	27	35	

Above values to be used only when beam has a top angle or side lug.
For unit stresses see sheet 8.13a.

seat angle. The seat angle is nevertheless more conservatively designed because the 24 kips per sq in. stress is the maximum allowed in bending. The 24 kips per sq in. in the beam-web is an allowance for average stress whereas the actual stress is variable and may pass the yield point locally at working loads.

The table on the following page compares the A.B.Co. or new A.I.S.C. allowable reaction values with the test results. The maximum values of test reaction are fairly uniform and it was noted that at this stage the beam webs had yielded due to general shear failure as well as by local crushing over the supports. The uniformity in result may also be due in part to the fact that one-half and three-fourths inch thick seat angles were weak in relation to the beam and the design was therefore unbalanced.

The maximum reactions were multiplied by 33/40 to adjust from the test yield point of 40 kips per sq in. to the specification yield point of 33 kips per sq in. for structural steel. The stiffened seat was the only support with enough nominal design strength to develop the full allowable beam reaction. The factor of safety of 1.63 for this connection is close to the A.I.S.C. tensile stress ratio of $\frac{33}{20} = 1.65$. The 6 by 4 by 1 in. seat angle gave practically as good results as the stiffened seat but is penalized by the arbitrary 35 kips top limit for unstiffened seats as specified by the A.I.S.C.

Reaction values are also given for the 0.04-in. permanent set in the top angles. These values are not so affected by beam shear failure as are the maximum test reactions. The factor of safety of 1.53 on this basis for the stiffened seat is below 1.65 by about ten per cent but in view of the reserve strength available this is hardly a serious difference.

The table of test results also includes two tests (W-9 and W-10) on 18 WF 47 beams, tested in another investigation[7]. These tests are of interest because local web yielding was followed by general web buckling as well as web crippling. The seat angles in these tests were welded and flexible top plates were used rather than top angles. The use of flexible top plates undoubtedly permits greater vertical deformation in the beam web and is therefore more conducive to web buckling than a stiffer top connection. Nevertheless the factor of safety was more than adequate in these tests, which again were too few in number to form a basis for general conclusion.

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7. "Tests Of Miscellaneous Types Of Welded Building Connections", B. Johnston and G. R. Deits, October 1941 A.W.S. Meeting.

Specimen	Seat Angle	Beam	Bearing Length	Allowable "R" for Rigid Support	AISC "R" Balanced Design	(R) Max by Test	R Maximum Adj. to 33 k.s.i. Yield Point	Adj. R Max. / R Allowable	R at 0.04-in. Top Angle Set	R at 0.04-in. Adj. to 33 ksi Yield Point	Adjusted R / Design R
ba	6 x 4 x 1/2 x 8-1/2	12WF 50	3-1/2	45.6	19.0	85.0	70.2	3.70	61.8	51.0	2.68
bc	6 x 4 x 3/4 x 8-1/2	12WF 50	3-1/2	45.6	28.0	85.0	70.2	2.50	66.8	55.1	1.97
bb	6 x 4 x 1 x 8-1/2	12WF 50	3-1/2	45.6	35.0*	87.5 90.0	72.2	2.06 2.12	82.2	67.8	1.94
bd	stiffened seat	12WF 50	3-1/2	45.6	45.6+	90.0 87.5	74.3	1.63 1.58	84.5	69.7	1.53
W- 9	6 x 3-1/2 x 5/8 x 9	18WF 47	3	33.6	23.0	75.0	57.6	2.51			
W-10	6 x 3-1/2 x 5/8 x 9	18WF 47	3	33.6	23.0	90.0	69.1	3.00			

* Not balanced design, 35K maximum allowed for unstiffened seat
+ Not balanced design, 45.6 kips allowable for rigid support

CONCLUSIONS RELATING TO DESIGN

Although the tests in themselves are an insufficient basis for design recommendations a study of the results as well as general design considerations leads the authors to the following observations relative to design procedure:

(1) On the basis of the few tests in this program an allowable average bearing stress of 23 kips per sq in. would appear permissible over the bearing distance "B + k". This is probably not enough different from the currently allowable value of 24 kips per sq in. to warran change without further test. Since there will be localized yielding at design loads the foregoing conclusion can be justified theoretically only on the basis of limit design.

(2) As now noted (and in the absence of other tests) the proposed design values should be used only in case a top or side angle is used.

(3) The tests in this investigation provide no information relative to what design limitations should be made, if any, with regard to the possibility of general web buckling as contrasted with local web crippling.

(4) The proposed A.B.Co. design procedure for seat angles has been included as a matter of reference. The following suggestions are made relative to the A.B.Co. design sheets.

(a) Limit the distance "x" to not less than T/3. This will insure against overstress in shear, as based on equating the bending strength at 24 kips per sq in. maximum stress with shear strength based on an average allowable stress of 13 kips per sq in.

(b) The statement "$B = \frac{R}{Q} - k$, but not less than $\frac{1}{2}$ (O-O)" could be more simply stated: "$B = \frac{R}{Q} -k$, but not less than k".

(c) The previous arbitrary upper limit of 35 kips for unstiffened seats has been carried over into the proposed table of design values, which is presumably less arbi rary than the previous specification. If the arbitrary limit were removed the design load for the seat would in any case be limited by the single shear value of the rivet group in the vertical leg. This amounts to 36.1 kips for four 7/8 in. rivets, at the A.I.S.C. allowable shear stress of 15 kips per sq in.

CONCLUSIONS RELATING TO TEST RESULTS

In addition to the foregoing comments on design procedure the following conclusions are summarized relative to the four tests which were made in this program.

(1) The relatively flexible 1/2 and 3/4-in. seat angles allowed much higher local stresses near the end of the beam web than did the 1-in. seat angle and the stiffened seat. The differences in stress distribution were probably influenced by accidental differences in contact as well as by differences in flexibility.

(2) In the case of the 1/2 and 3/4-in. seat angles, tested with the top angle unbolted, the stress distribution in the web in the elastic range was similar to the theoretical stress distribution for a point load at the end of the beam.

(3) In each of the tests the length of the plastic zone of beam web yield measured from the end of the beam approached the length "B + k" now used as a design criterion.

(4) The bolted top angles participated in carrying the vertical reaction. At low loads the participation is greater than would be inferred by the relative stiffness because the top angles deflect more than the seat angles. This is due to the added deformation in the beam web.

(5) At the maximum load each of the specimens showed a marked yielding or crushing of the beam web above the supports, as indicated by the flaking away of the whitewashed mill scale. There was also a general yielding in the beam webs due to shear at loads near the maximum. The general progress of failure in all of the specimens was gradually progressive in nature.

(6) Except for slight yielding of the thinnest seat angle there was little sign of failure in the seat angles or stiffened seat at the maximum load.

(7) There was no indication of web buckling in any of these tests, but the tests do not include web depth-thickness ratios for which such failure would be likely.

TABLE I

PHYSICAL PROPERTIES OF MATERIAL

Source	Per Cent * Elongation in 2 in.	Stress in kips/sq in.	
		Upper Yield Point	Ultimate
Beam web, with direction of rolling, near center	40.5	43.5	64.6
Beam web, with direction of rolling, near fillets	49.0	38.0	60.5
Beam web, 45° to direction of rolling	43.7	41.3	61.6
Beam web, across direction of rolling	40.5	42.4	62.6
Beam flange	59.2	36.0	58.3
1/2-in. seat angle, ba	51.5	39.2	66.0
3/4-in. seat angle, bc	62.5	31.9	51.4
1-in. seat angle, bb	59.2	33.8	64.0
3/8-in. stiffened seat, bd	53.5	36.1	58.5
5/16-in. stiffener (for bd)	48.5	37.4	58.0
Top angle	47.0	40.3	67.0
Column flange	58.0	34.4	55.0

* Note, cross-sectional size and shape is variable, hence there is no direct relation between various per cent elongation values

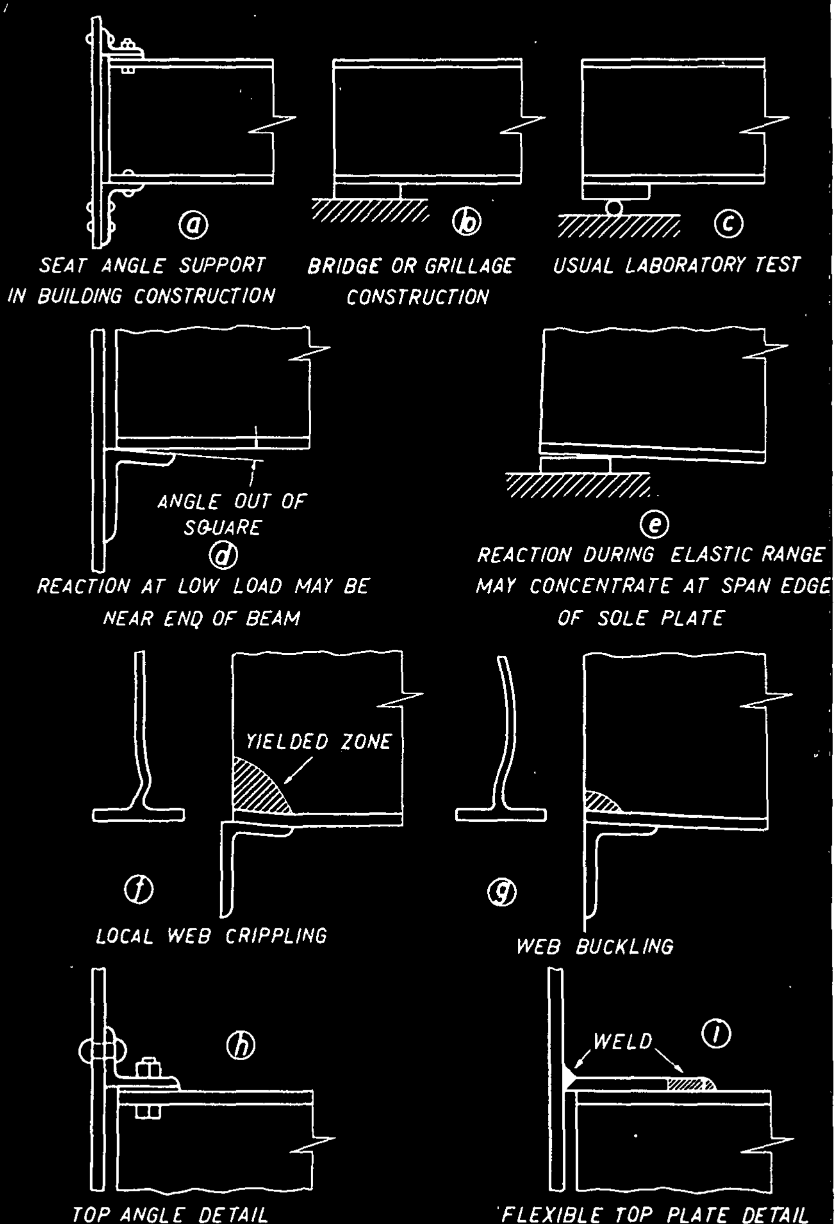


FIG. 1 - GENERAL BEHAVIOR AT END OF BEAMS

7/82/23-7A

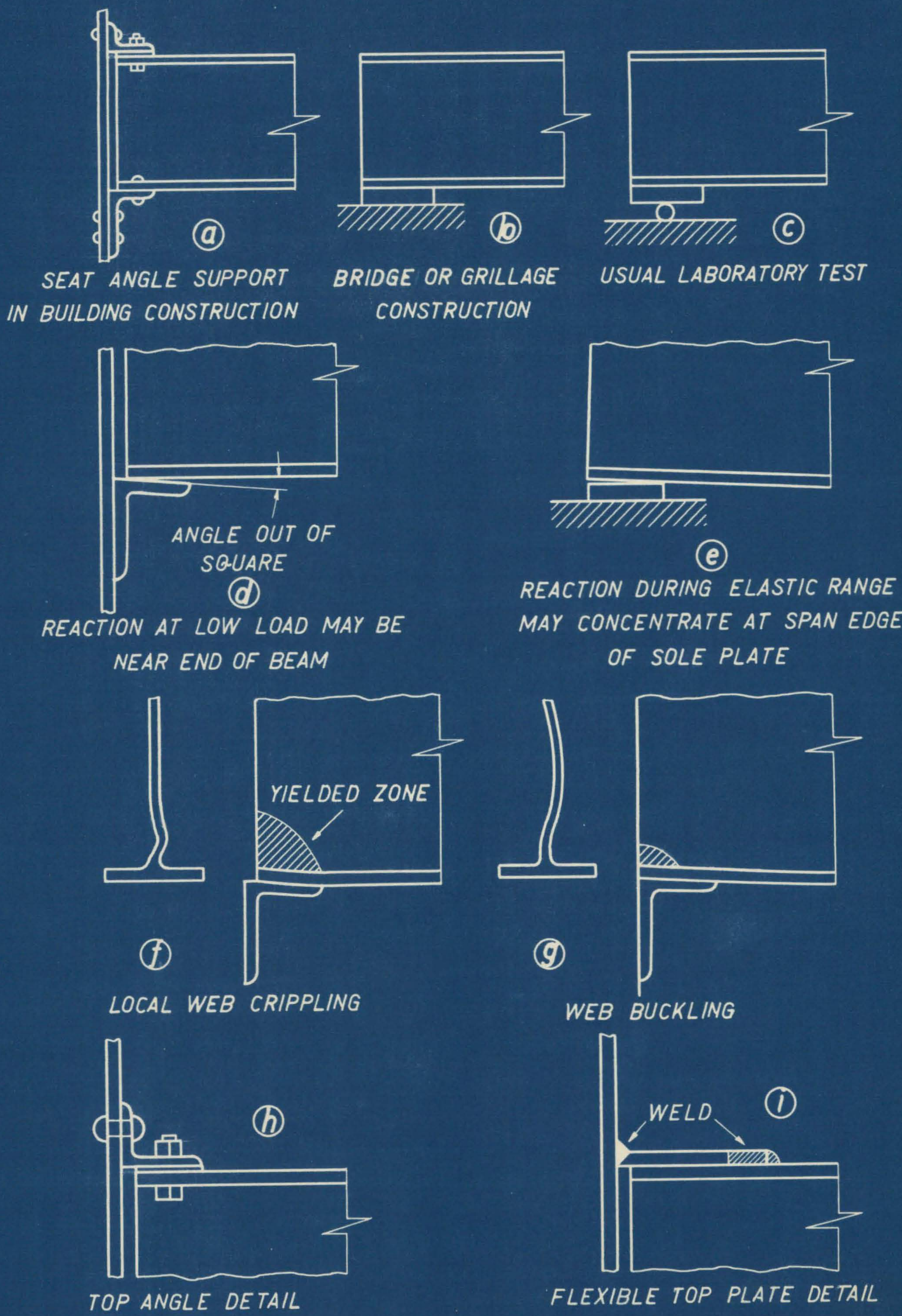


FIG. 1 - GENERAL BEHAVIOR AT END OF BEAMS

Negative No. 7/82/23-74

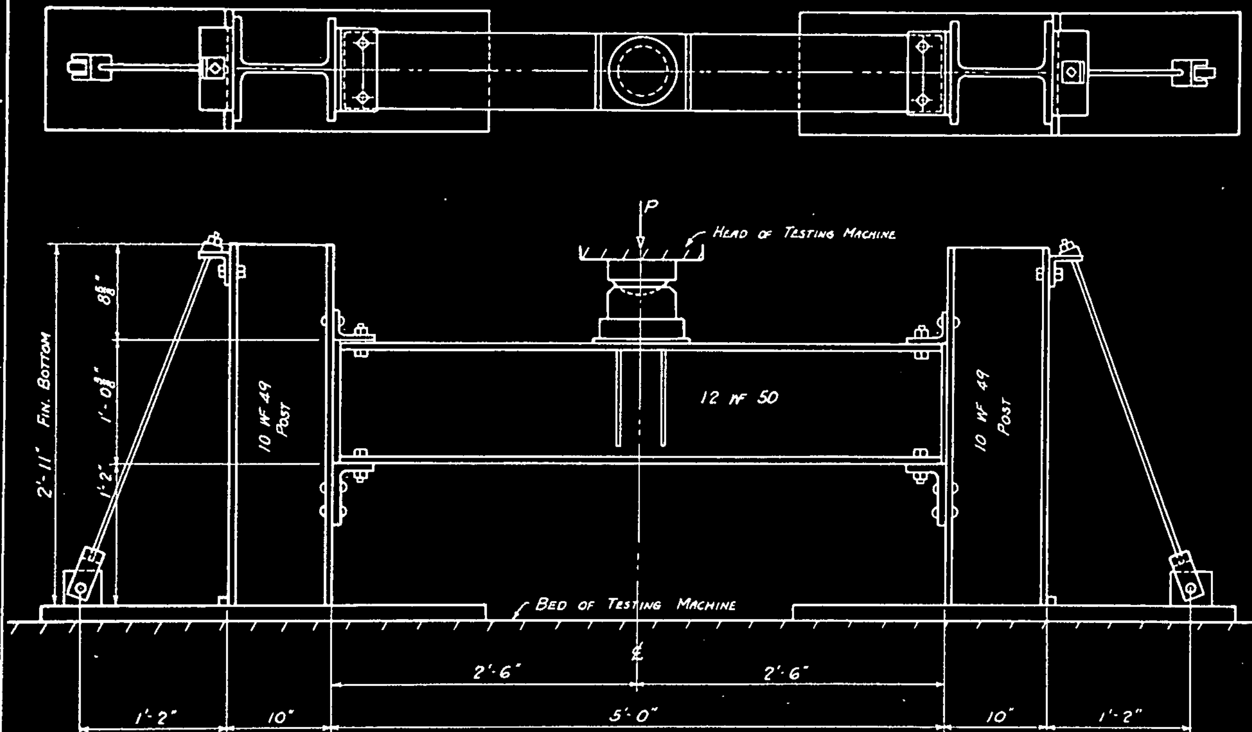


FIG. 2

SET-UP FOR WEB CRIPPLING TEST

7/82/23-8A

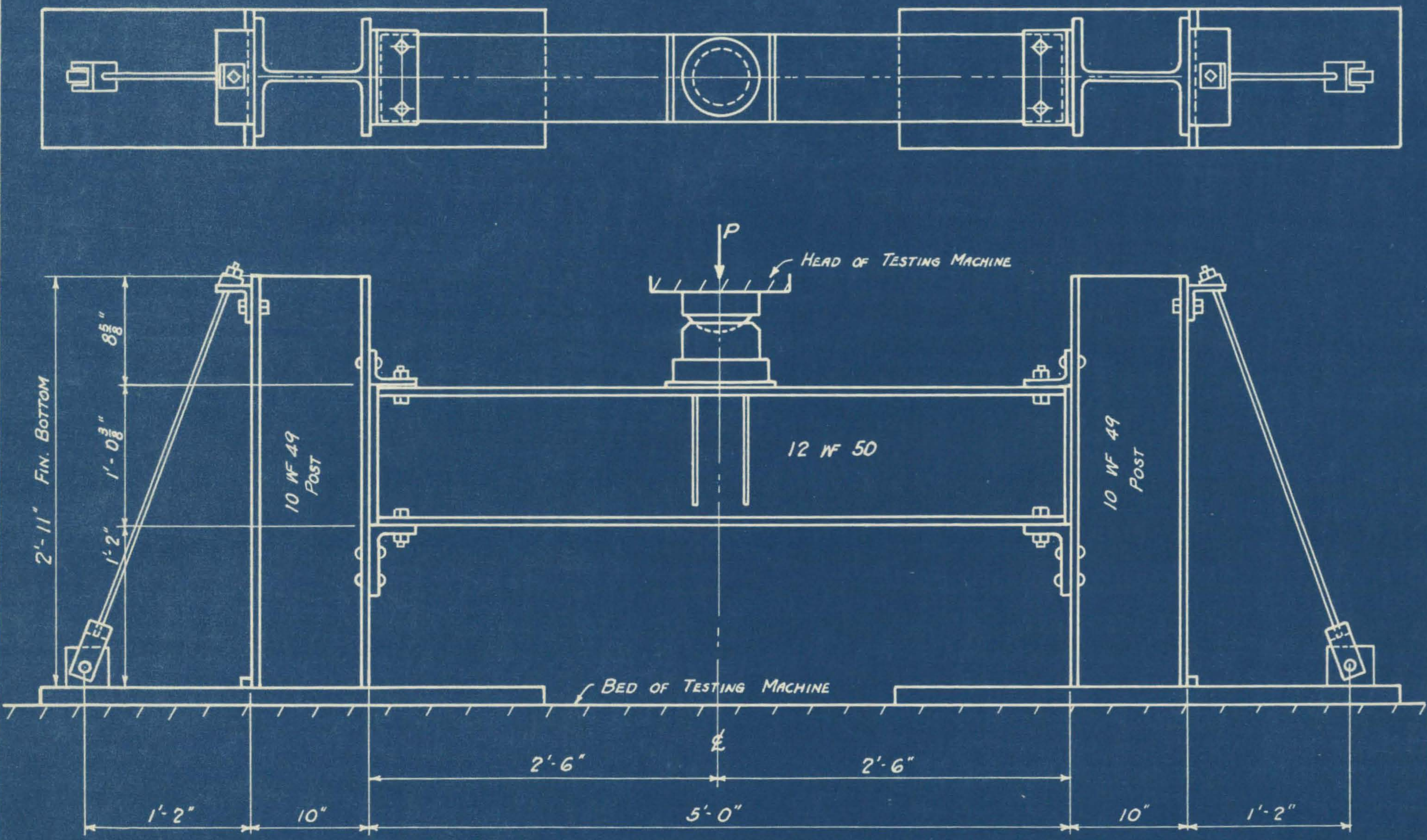


FIG. 2

SET-UP FOR WEB CRIPPLING TEST

Negative # 7/82/23-8A

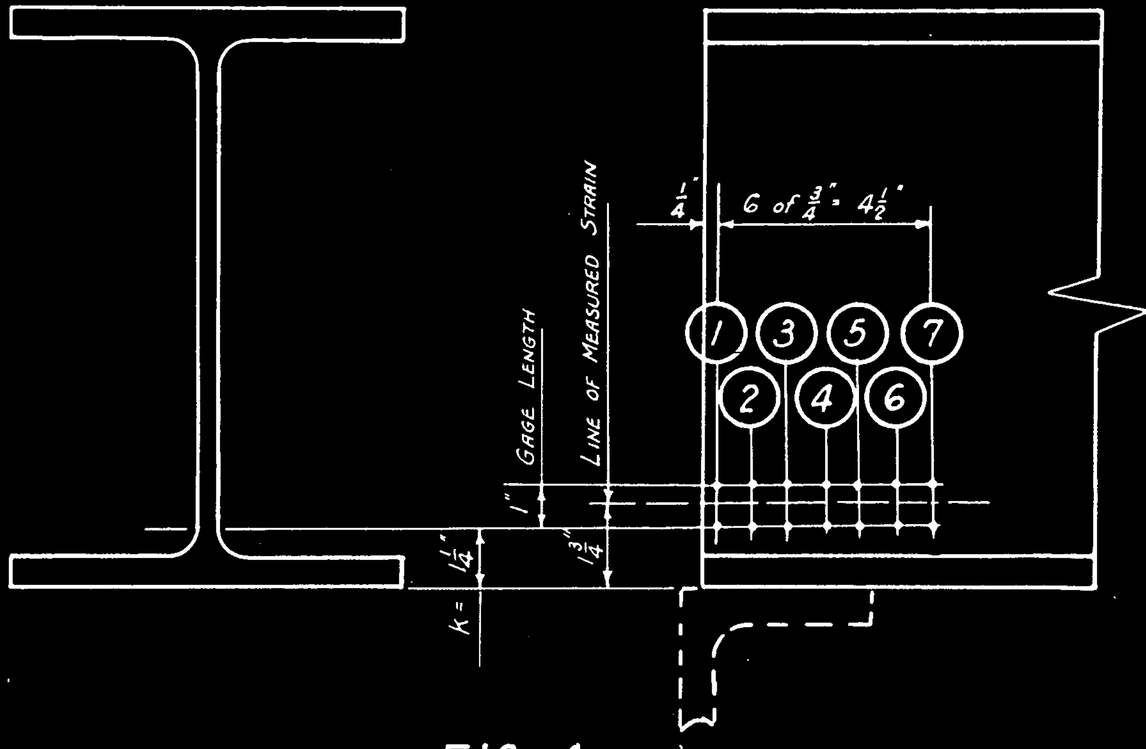


FIG. 4 a

LOCATION OF HUGGENBERGER GAGE SETTINGS - VERTICAL WEB STRAIN

7/82/23-17A

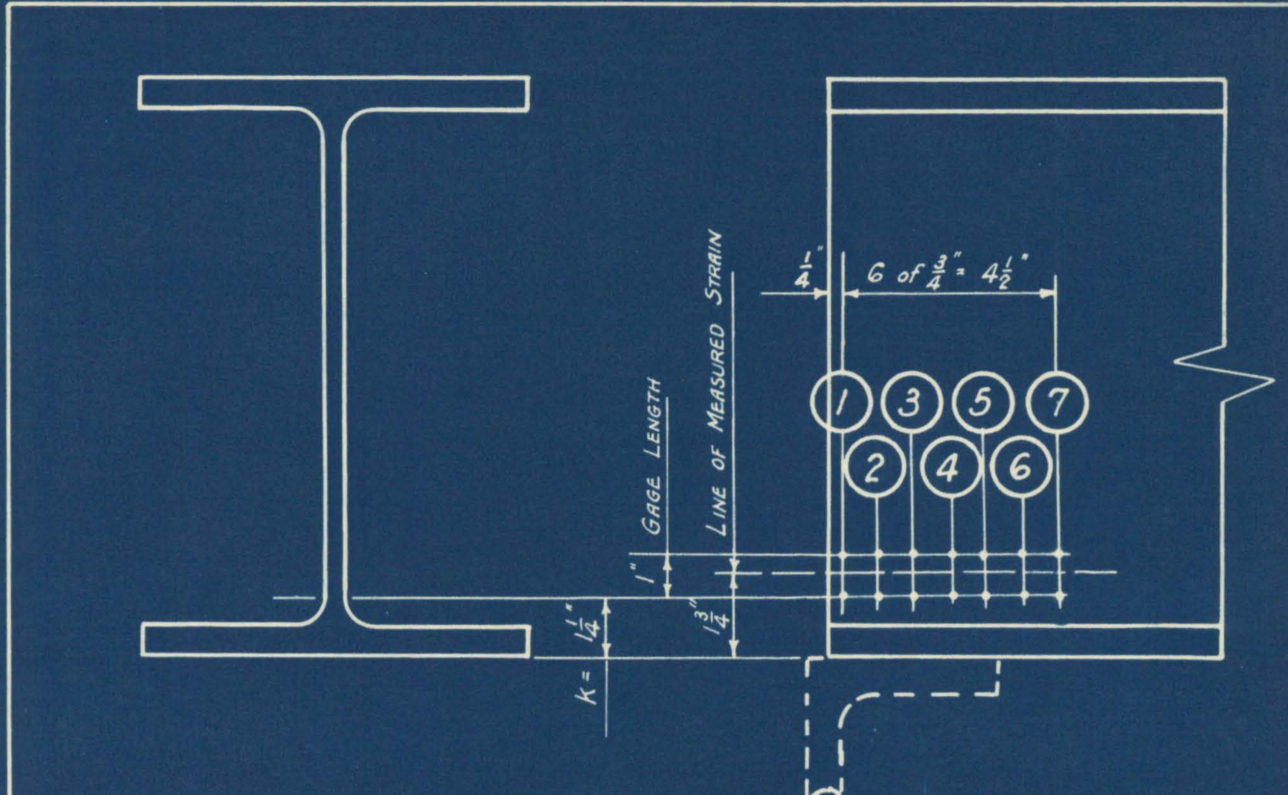


FIG. 4 a

LOCATION OF HUGGENBERGER GAGE SETTINGS - VERTICAL WEB STRAIN

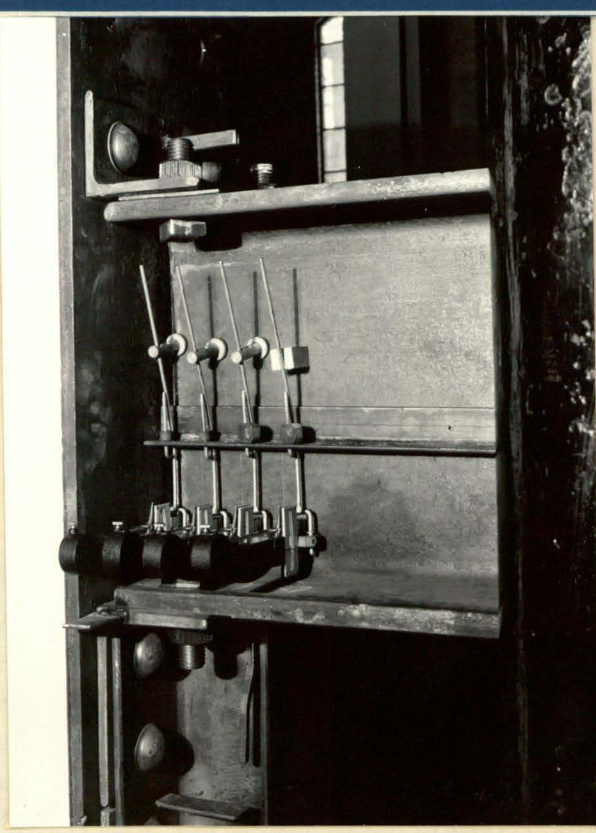


Fig. 4b
Vertical
Strain Measurements

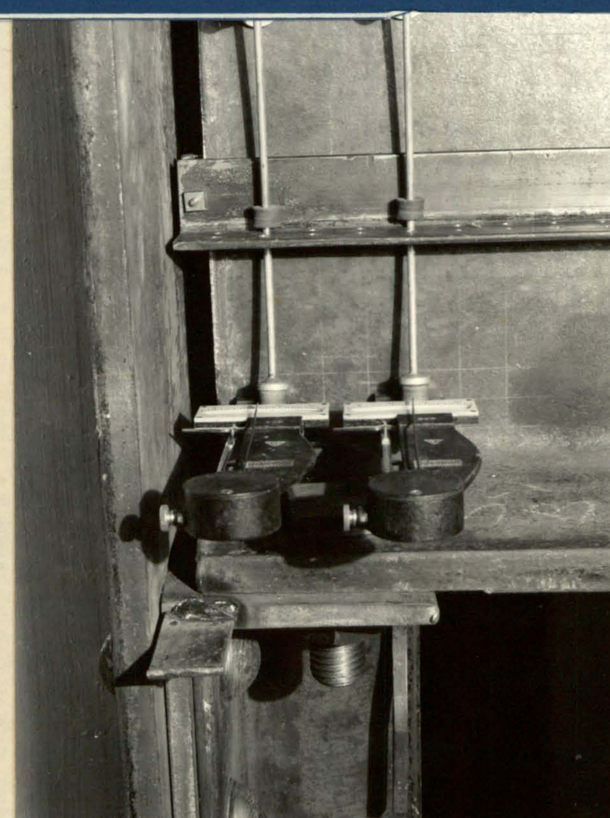


Fig. 4c
Horizontal
Strain Measurements

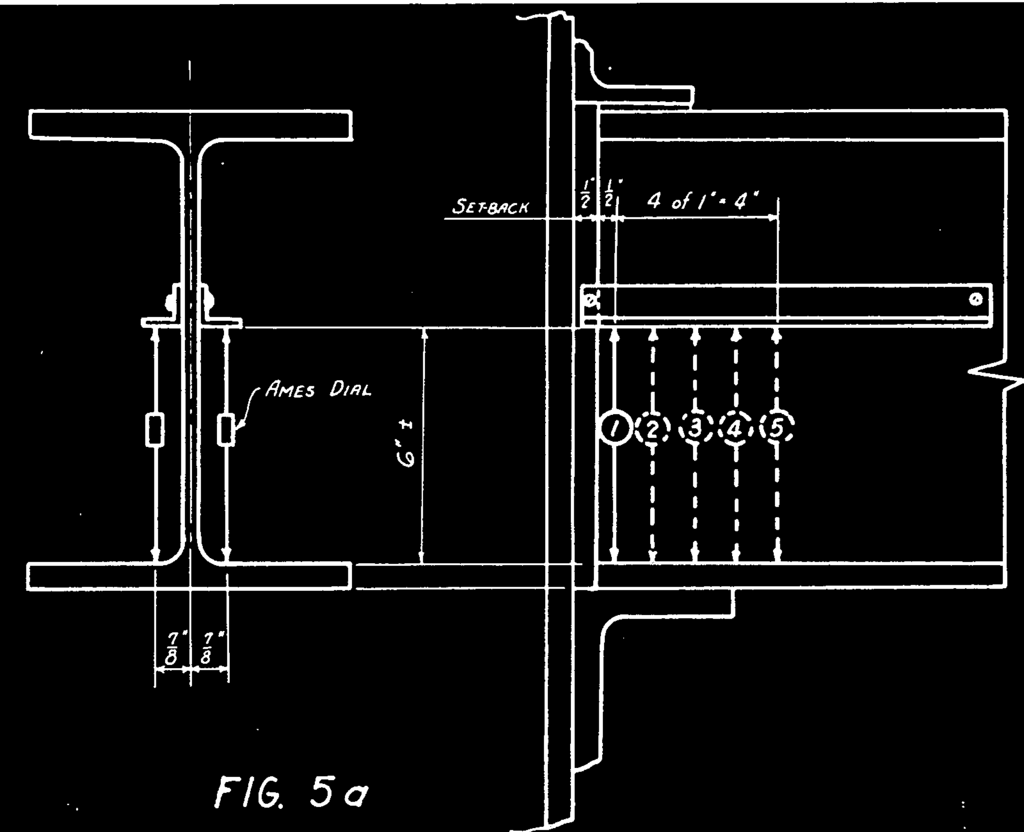


FIG. 5a

LOCATION OF STATIONS - MEASUREMENT OF COMPRESSION OF WEB

7/82/23-18A

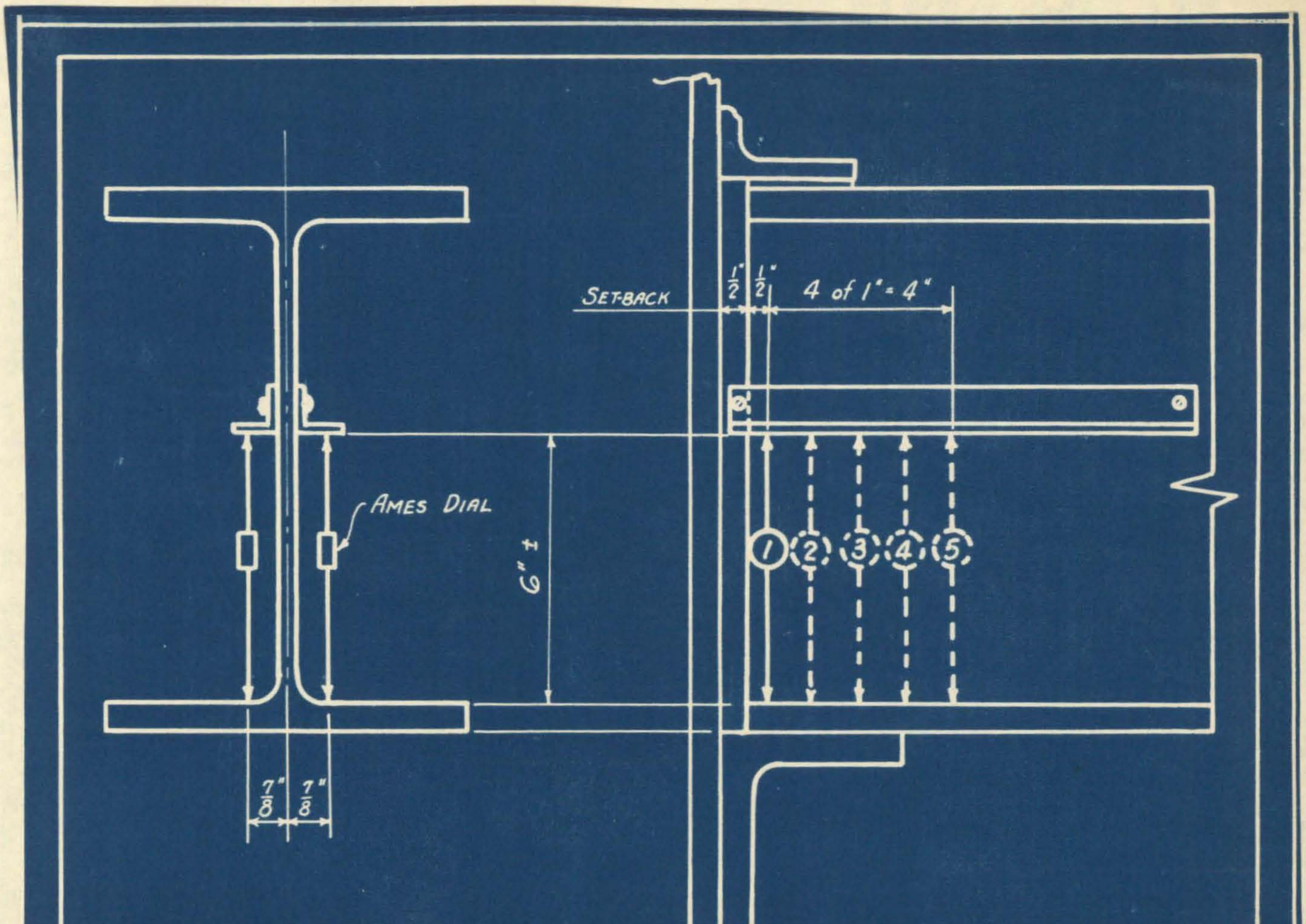


FIG. 5a

LOCATION OF STATIONS - MEASUREMENT OF COMPRESSION OF WEB

Fig. 5b
Measuring
Vertical
Deformation
in Beam Web

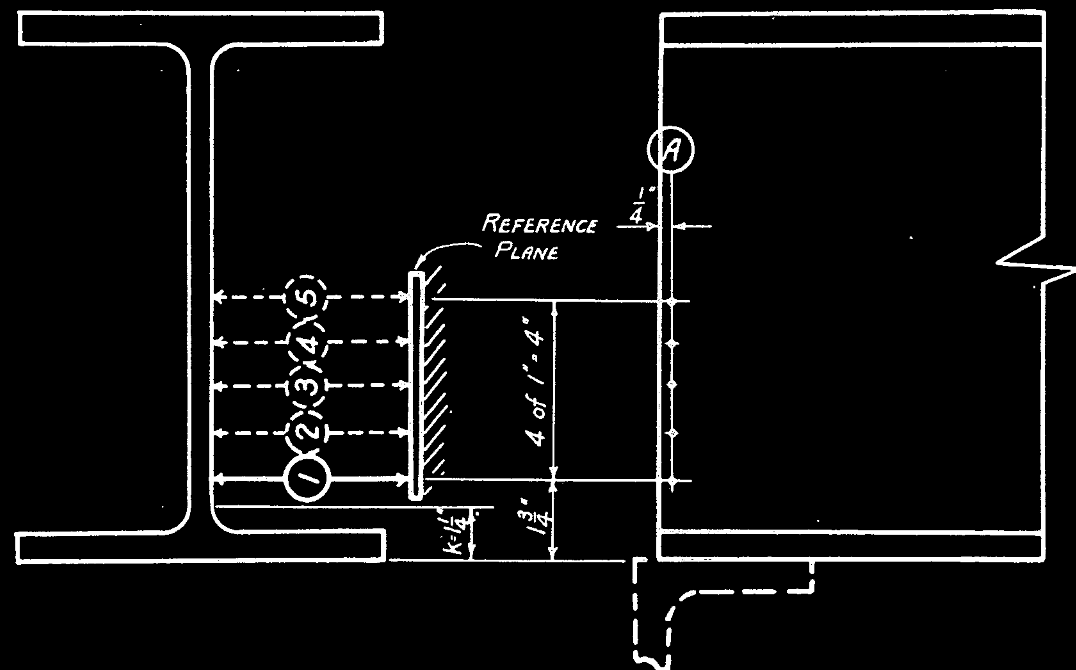


FIG. 6a

LOCATION OF STATIONS - LATERAL BUCKLING OF WEB

7/82/23-19A

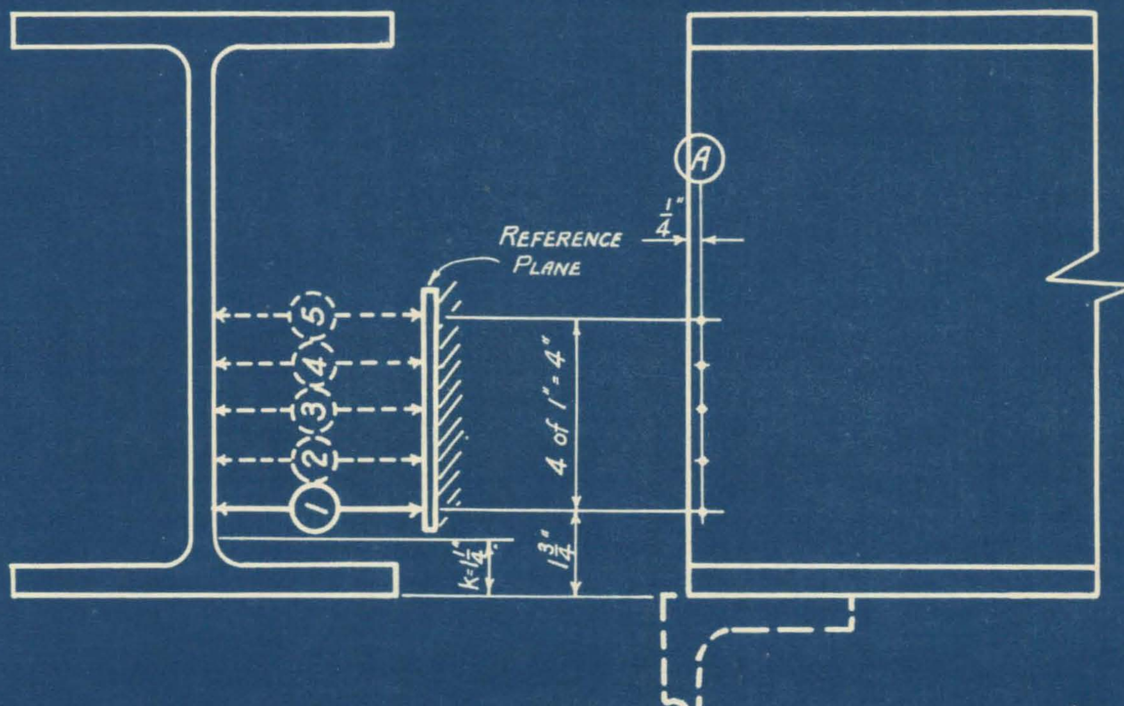


FIG. 6a
LOCATION OF STATIONS - LATERAL BUCKLING OF WEB

Fig. 6b
Measuring
Lateral Deformation
of Web

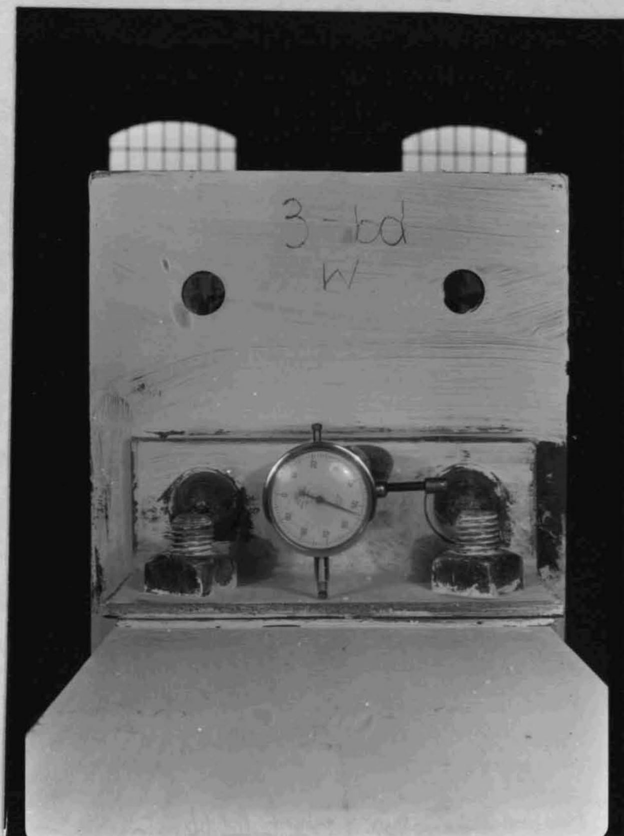


Fig. 7a
Top Angle
Deflection Measurement

Fig. 7b (Test bb)
Seat Angle
Deflection Measurement

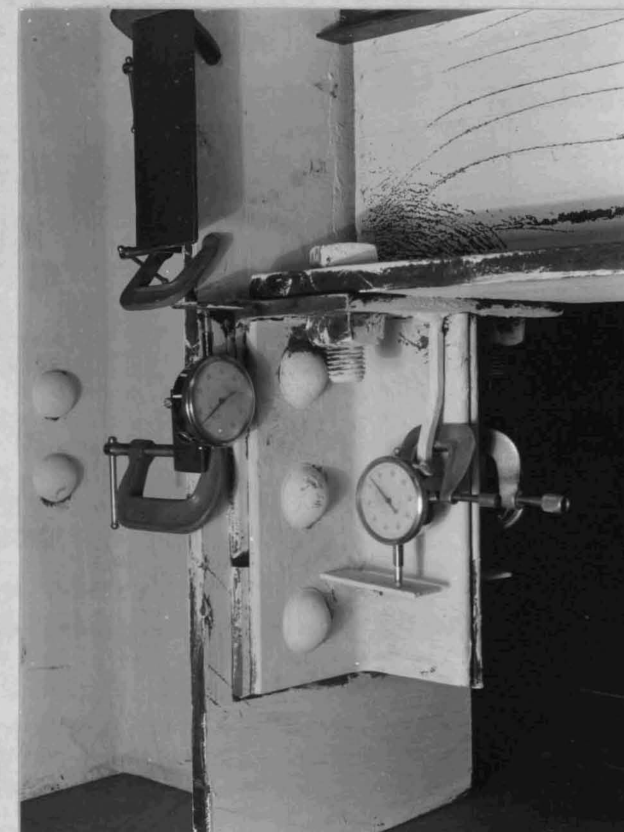


Fig. 7c (Test bd)
Stiffened Seat
Deflection Measurement

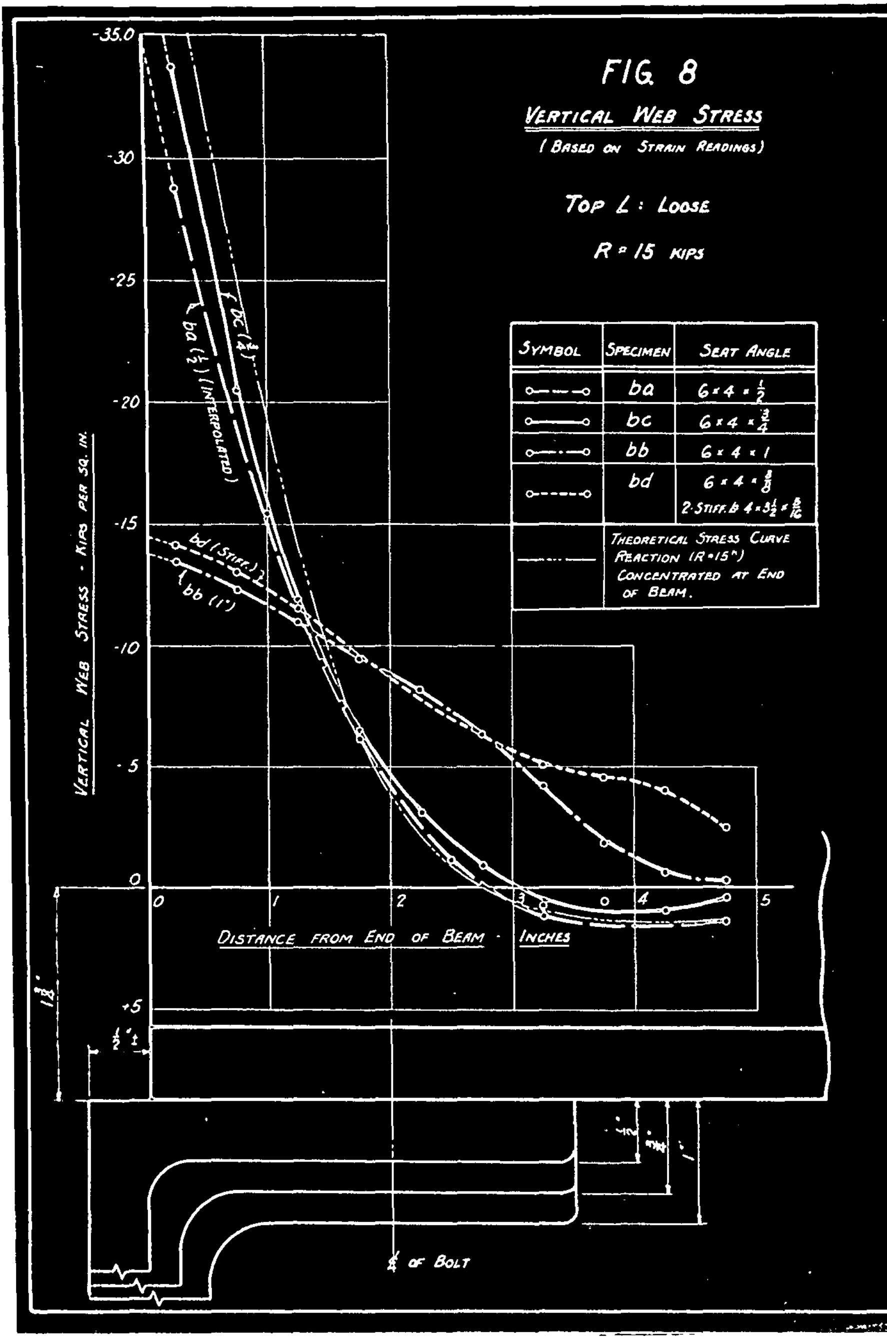


FIG. 8
VERTICAL WEB STRESS
(BASED ON STRAIN READINGS)
TOP ∠ : LOOSE
R = 15 KIPS
SYMBOL | SPECIMEN | SEAT ANGLE
ba | 6 × 4 × 1/2
bc | 6 × 4 × 3/4
bb | 6 × 4 × 1
bd | 6 × 4 × 5/8 2-STIFF. ∠ 4 × 3 1/2 × 5/16
THEORETICAL STRESS CURVE REACTION (R = 15ᵏ) CONCENTRATED AT END OF BEAM.
VERTICAL WEB STRESS - KIPS PER SQ. IN.
DISTANCE FROM END OF BEAM - INCHES
bc (3/4)
ba (1/2) (INTERPOLATED)
bd (STIFF.)
bb (1")
-35.0
-30
-25
-20
-15
-10
-5
0
+5
0
1
2
3
4
5
℄ OF BOLT

7/82/23-4A

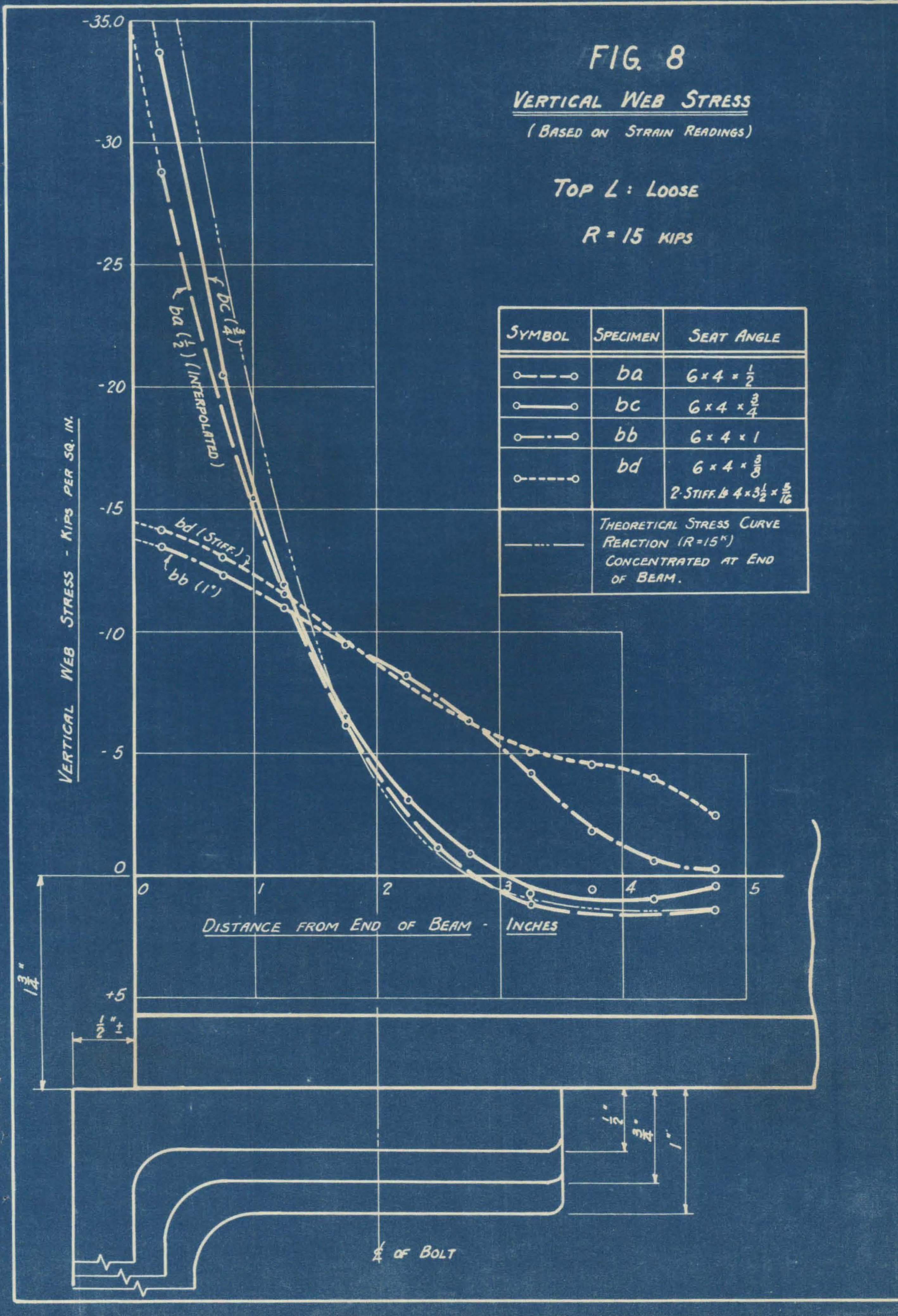


FIG. 8

VERTICAL WEB STRESS

(BASED ON STRAIN READINGS)

TOP L: LOOSE

$R = 15$ KIPS

Negative # 7/82/23-4A

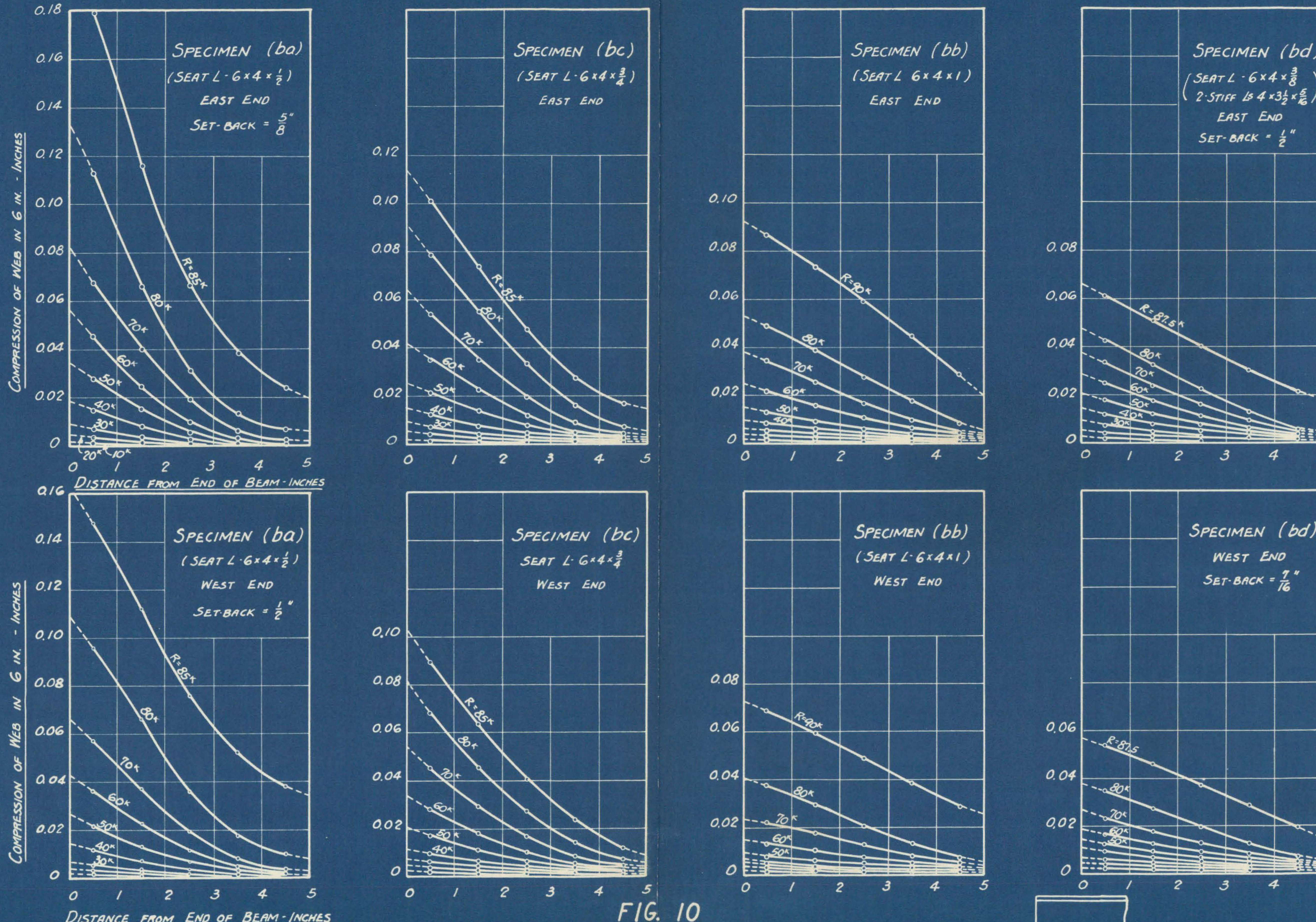


FIG. 10

COMPARISON OF COMPRESSION OF BEAM WEB

12 WF 50 TOP L: TIGHT

Negative # 7/82/23-12A

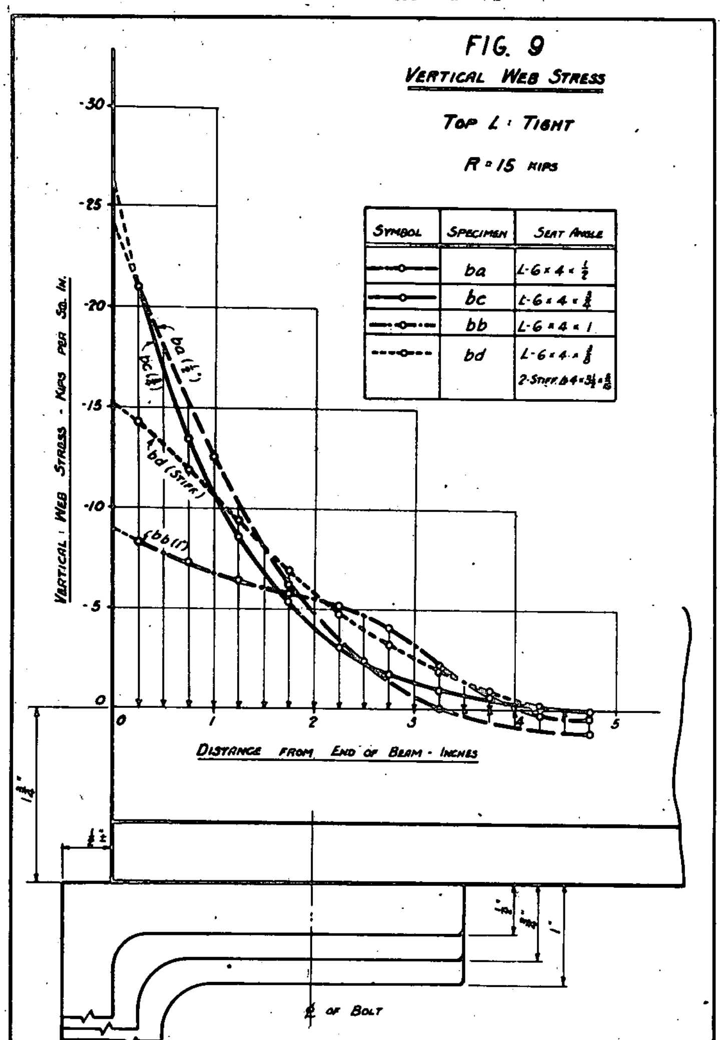


FIG. 9
VERTICAL WEB STRESS
TOP L: TIGHT
R = 15 KIPS
SYMBOL
SPECIMEN
SEAT ANGLE
ba
L-6×4×1/2
bc
L-6×4×3/4
bb
L-6×4×1
bd
L-6×4×5/8
2-STIFF. L 4×3½×5/8
VERTICAL: WEB STRESS - KIPS PER SQ. IN.
-30
-25
-20
-15
-10
-5
0
0
1
2
3
4
5
DISTANCE FROM END OF BEAM - INCHES
ba (½")
bc (¾")
bd (STIFF.)
bb (1")
℄ OF BOLT

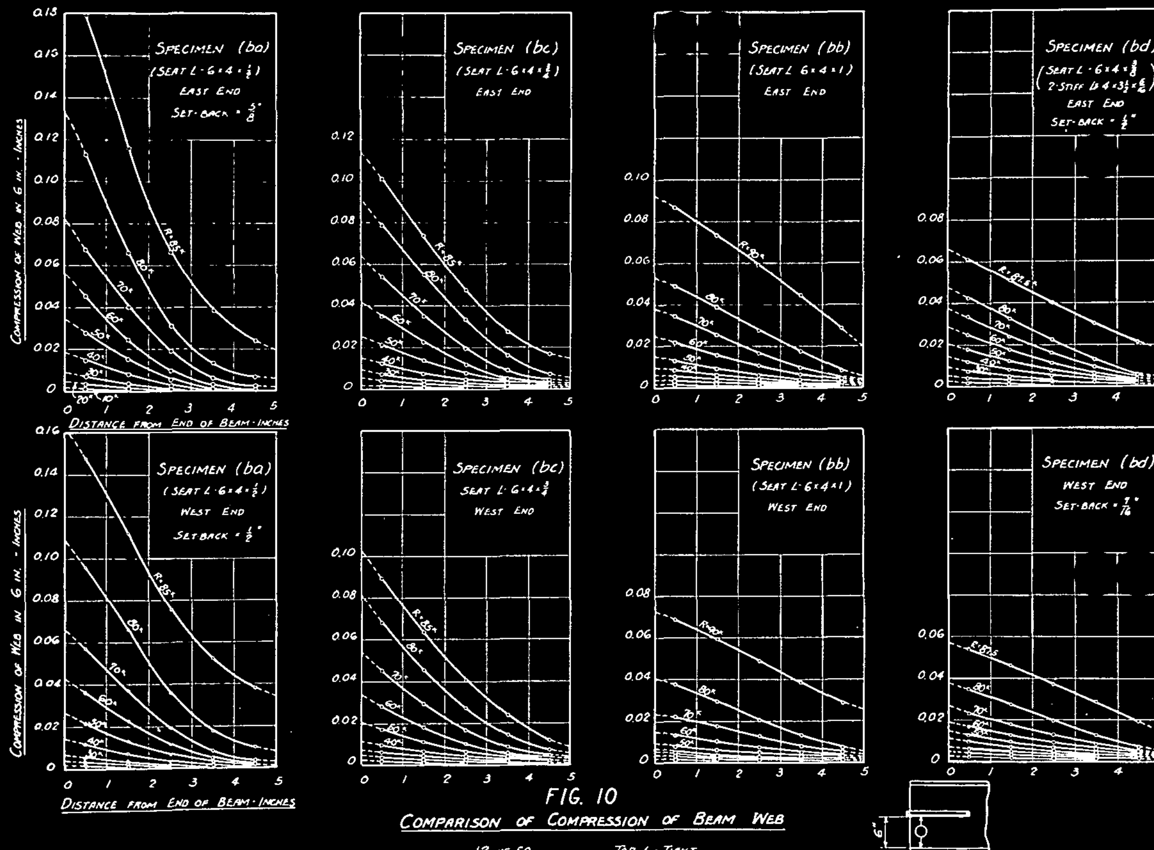
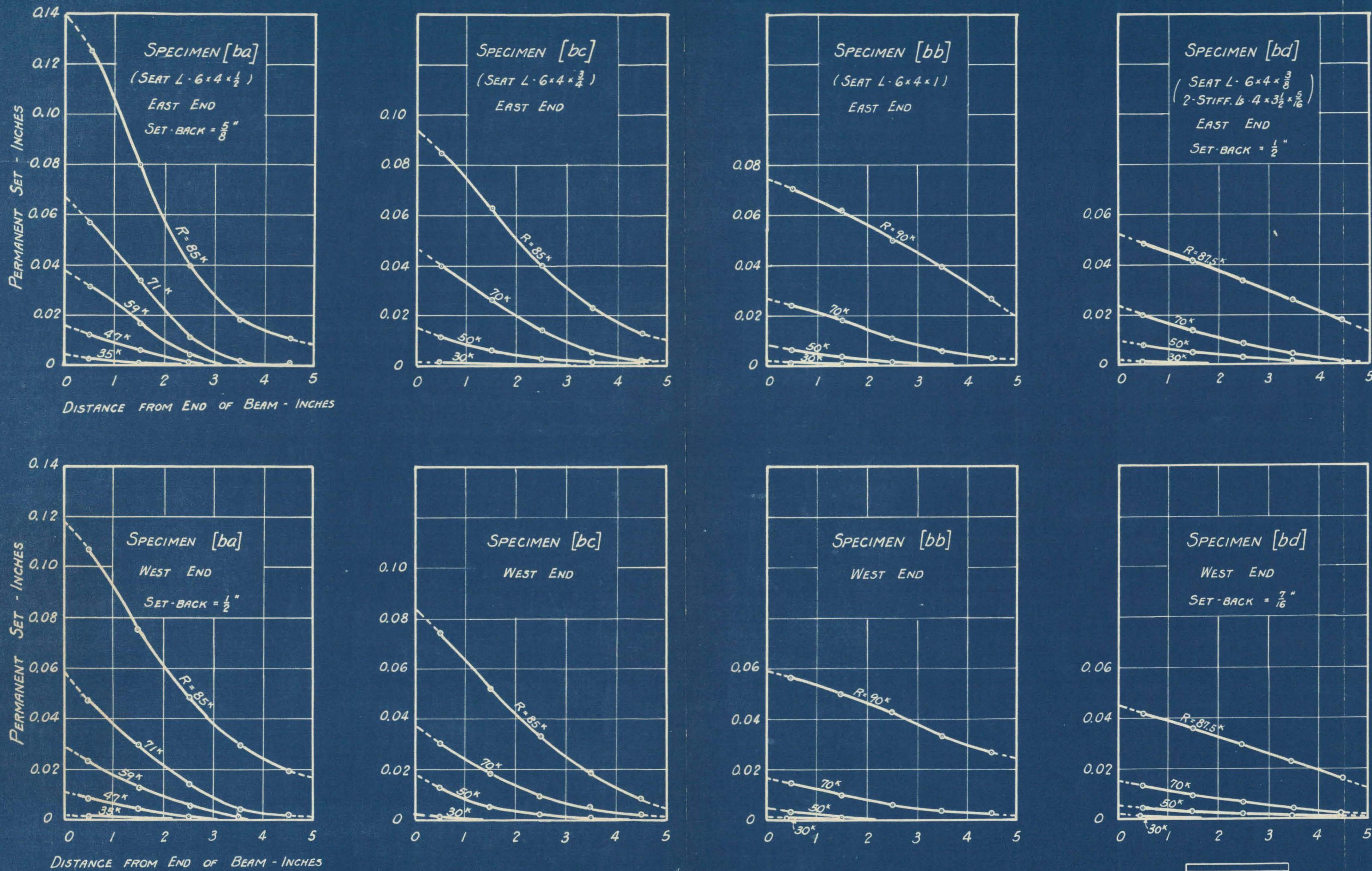


FIG. 10

COMPARISON OF COMPRESSION OF BEAM WEB

12 of 50 Top L: Tight

7/82/23-12A



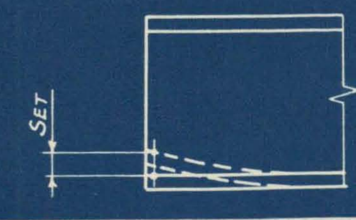


FIG. II

COMPARISON OF PERMANENT SET IN BEAM WEB

TOP ANGLE BOLTED

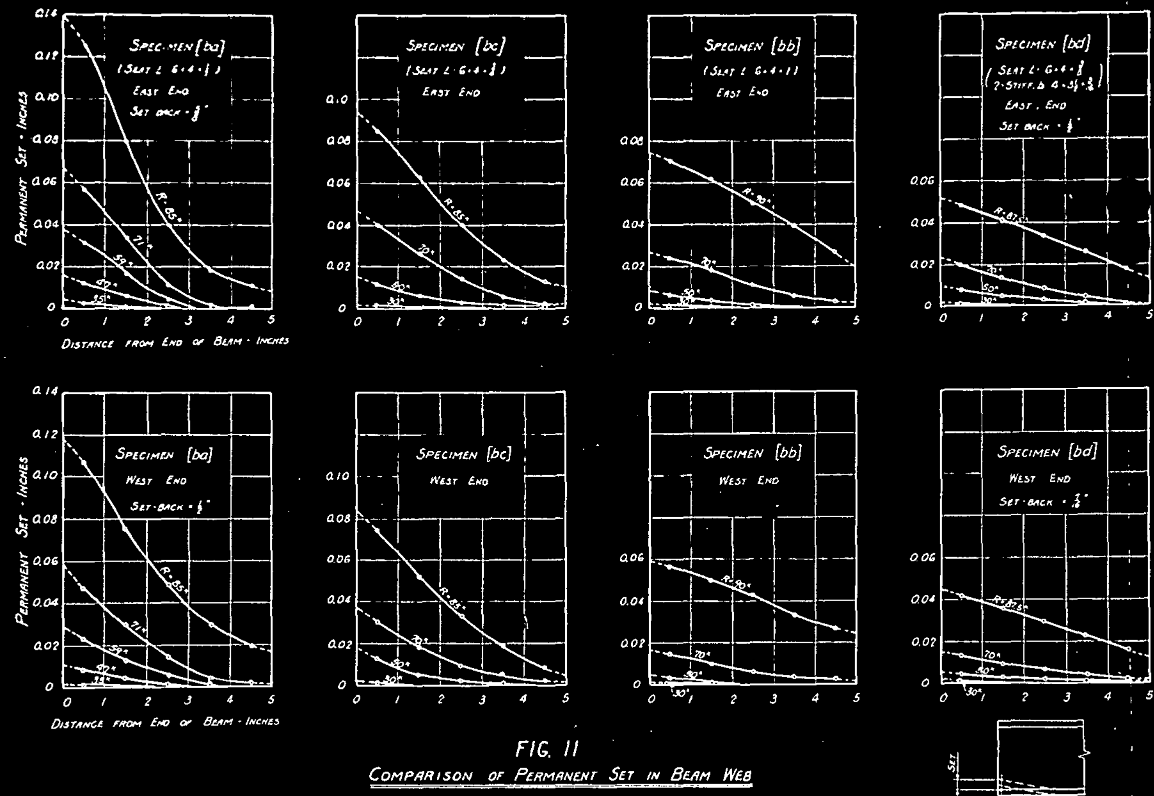
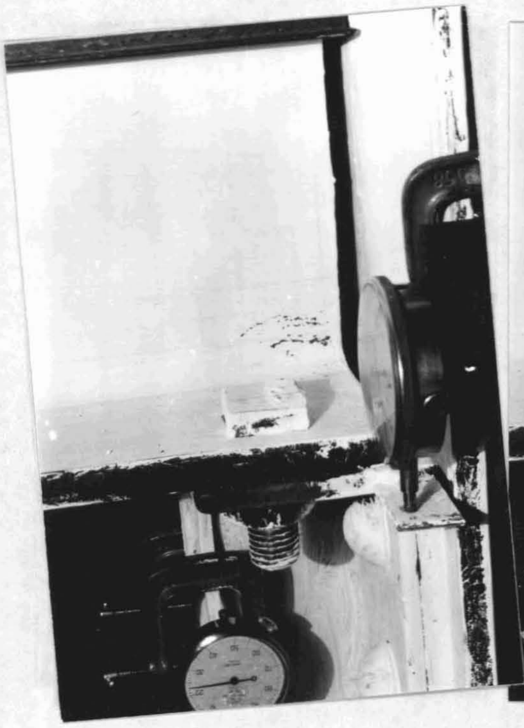


FIG. 11

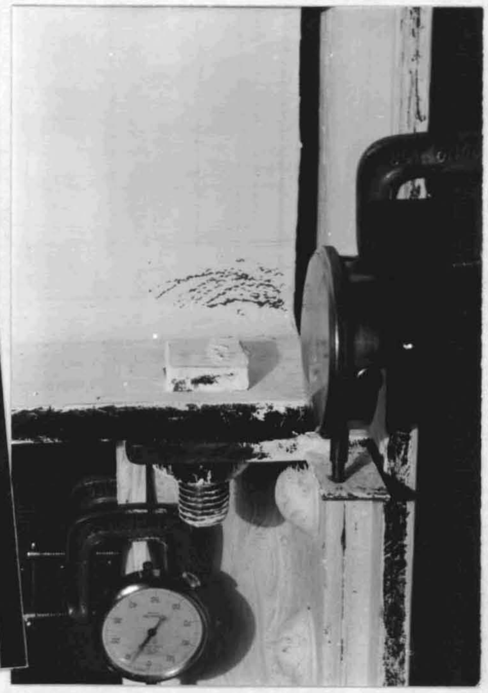
COMPARISON OF PERMANENT SET IN BEAM WEB

TOP ANGLE BOLTED

7/82/23-11A

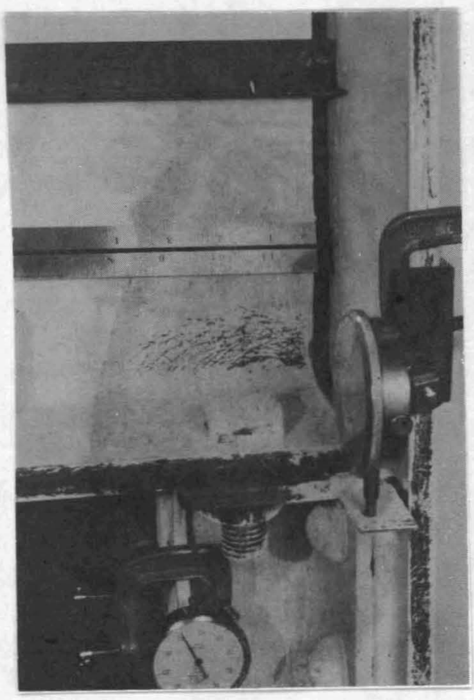


40 kips



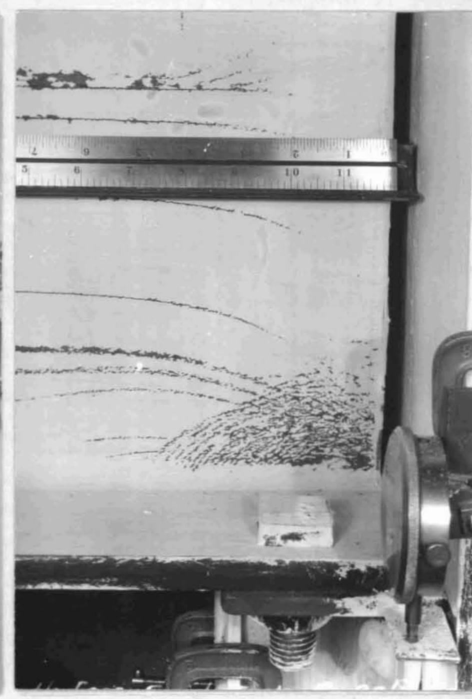
50 kips

60 kips



70 kips Fig 10

80 kips



85 kips

Fig. 12 - Test bd, Showing Progressive Yielding in Web of Beam

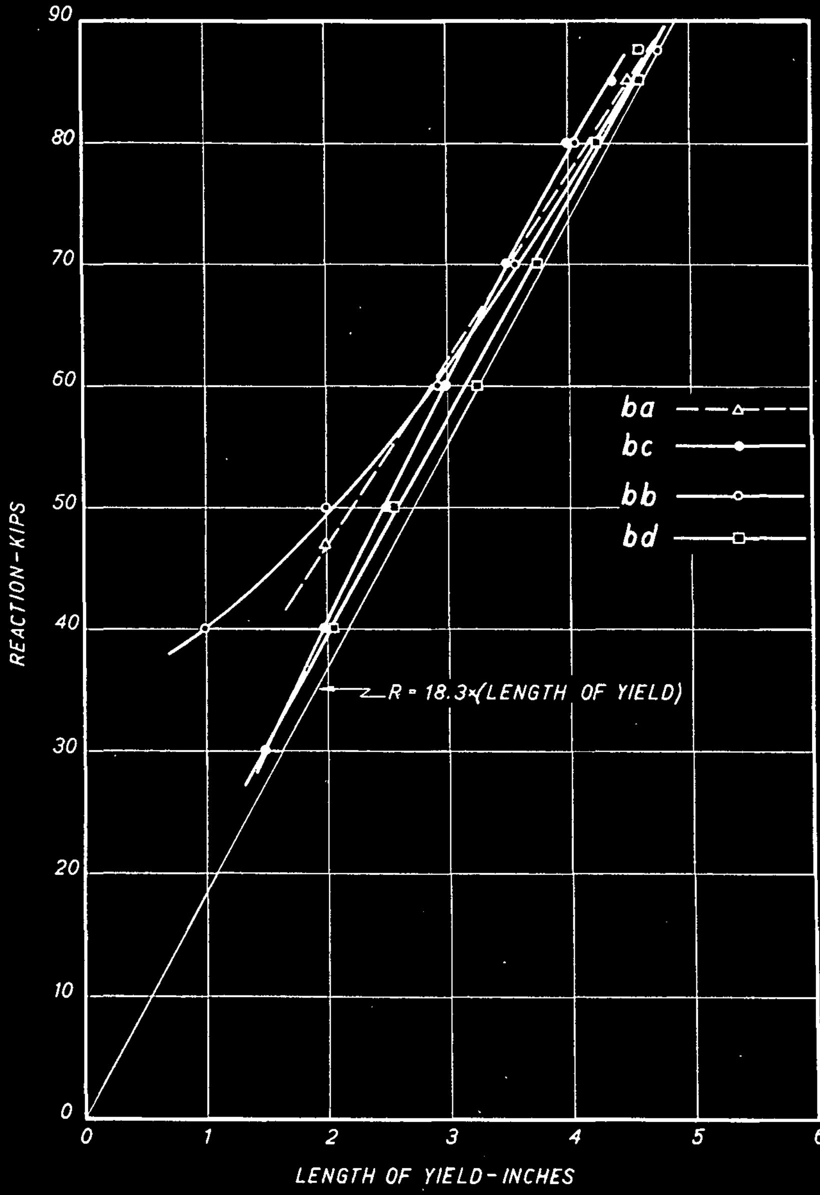


FIG. 13 RELATION BETWEEN REACTION LOAD AND LENGTH OF YIELD

7/82/23-5A

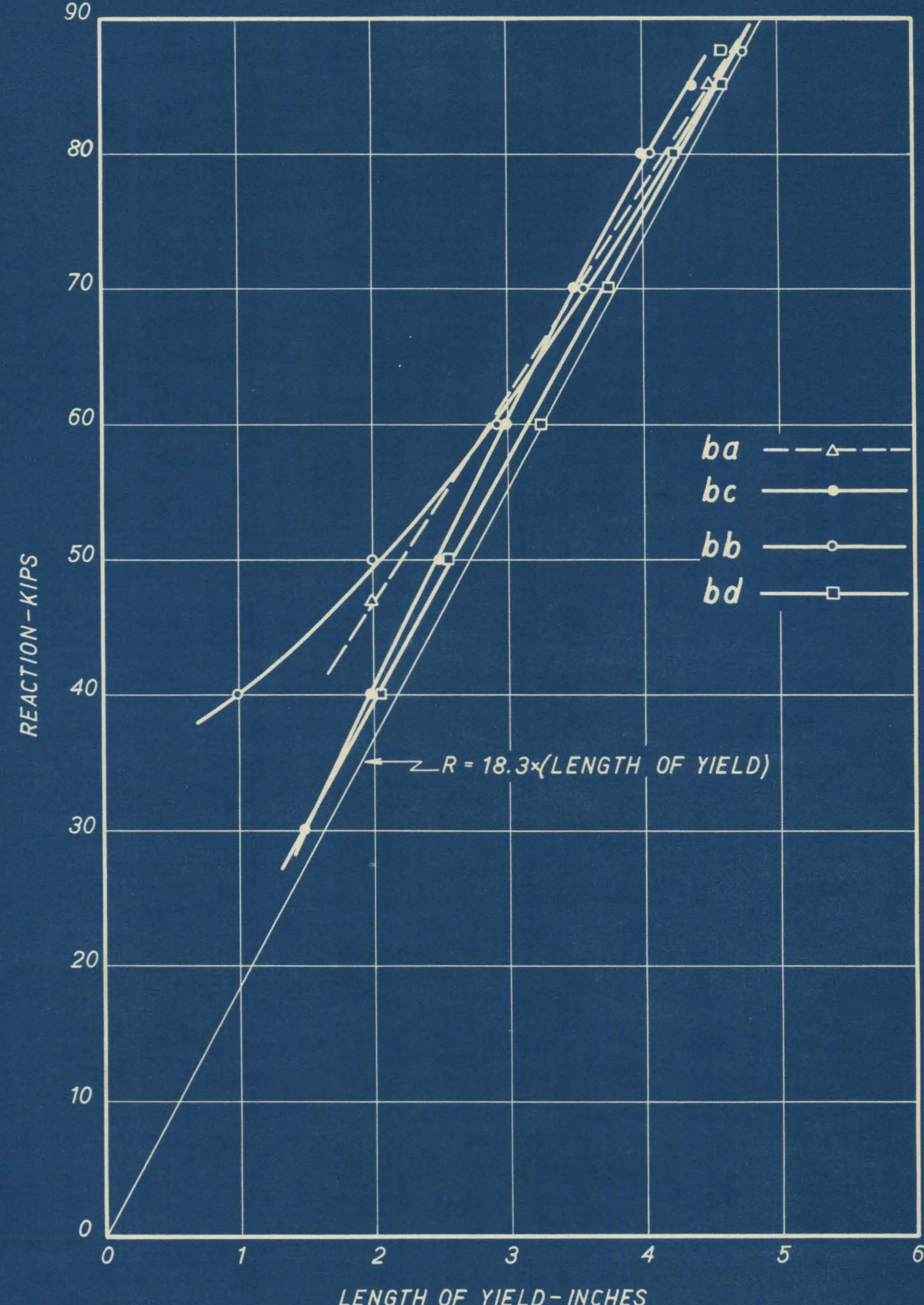
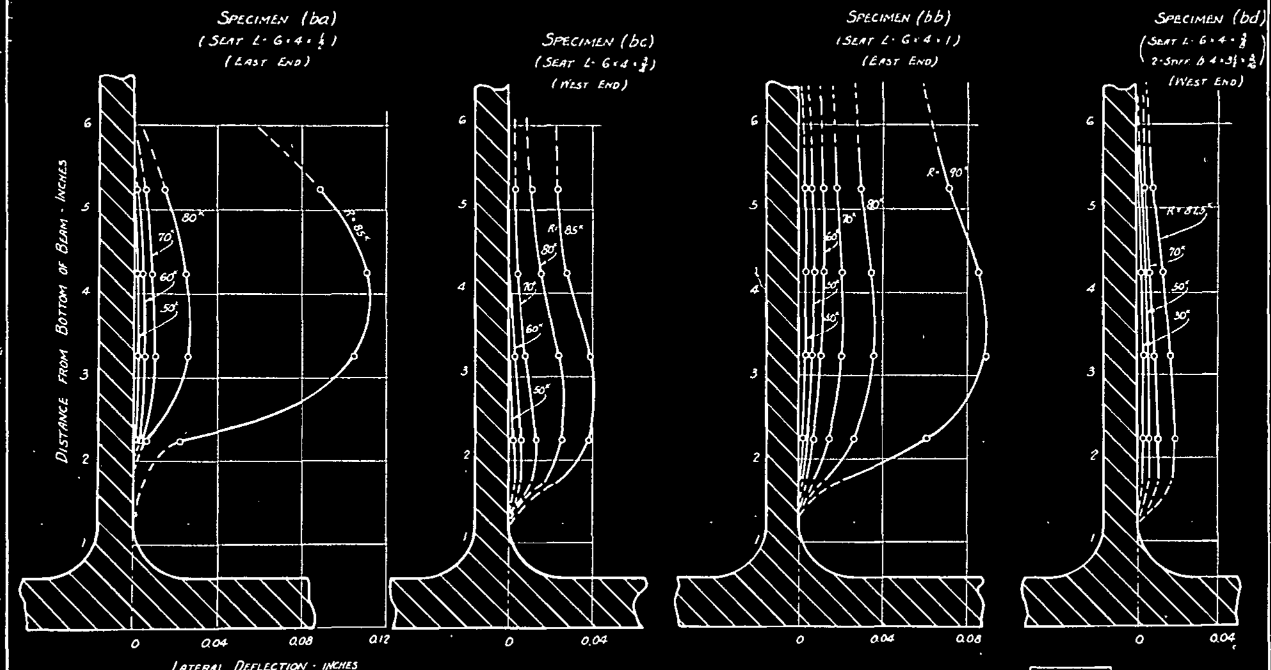


FIG. 13 RELATION BETWEEN REACTION LOAD AND LENGTH OF YIELD

Negative # 7/82/23-5A



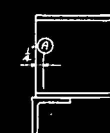


FIG. 14

COMPARISON OF LATERAL DEFLECTION OF BEAM WEB

(12 W 50)   TOP L - TIGHT   LINE (A)

7/82/23-10A

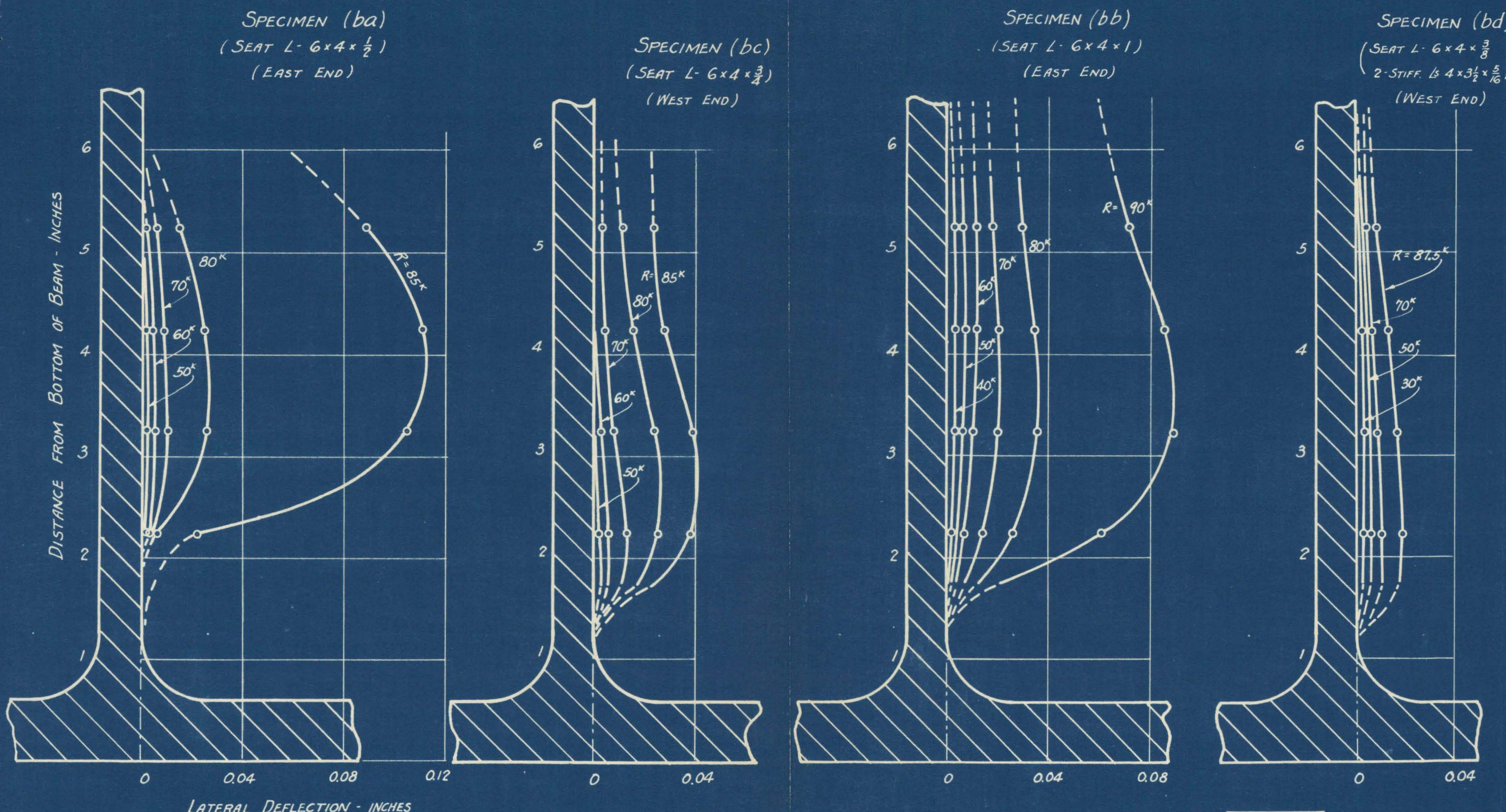
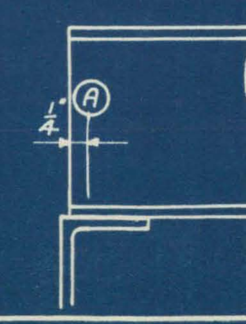


FIG. 14

COMPARISON OF LATERAL DEFLECTION OF BEAM WEB

(12 WF 50) Top L - Tight Line (A)



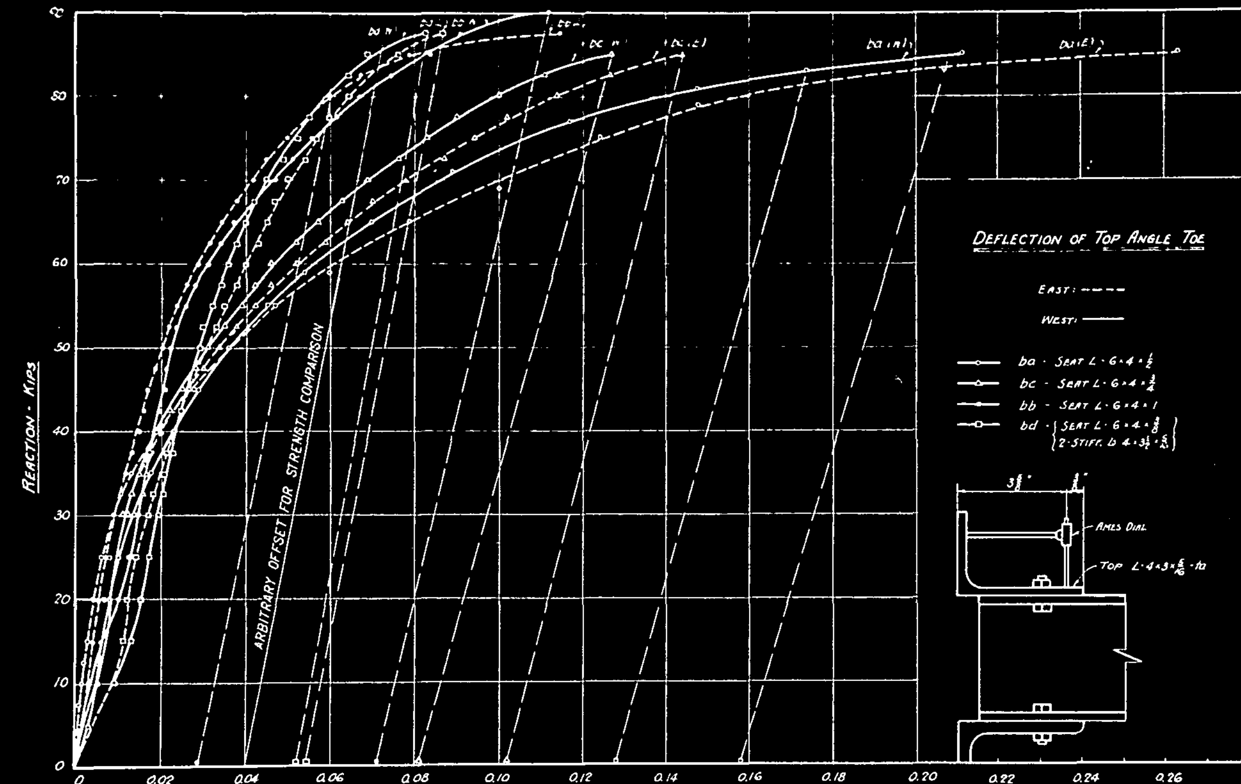
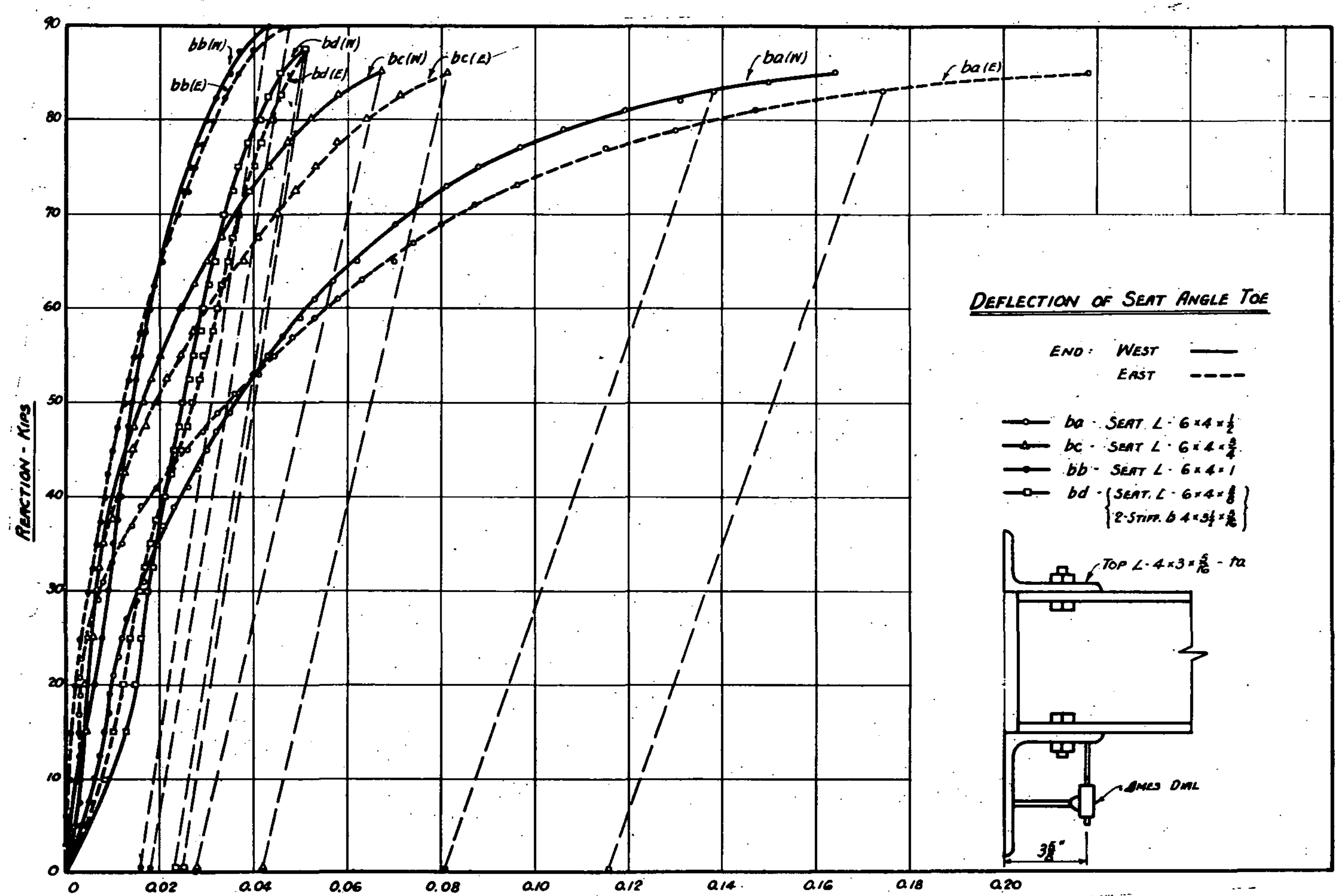


FIG. 16 DEFLECTION OF TOP ANGLE TOE - INCHES

7/82/23-9A



REACTION - KIPS
DEFLECTION OF SEAT ANGLE TOE - INCHES
bb(W)
bb(E)
bd(W)
bd(E)
bc(W)
bc(E)
ba(W)
ba(E)
DEFLECTION OF SEAT ANGLE TOE
END: WEST
EAST
ba - SEAT ∠ - 6×4×½
bc - SEAT ∠ - 6×4×¾
bb - SEAT ∠ - 6×4×1
bd - {SEAT ∠ - 6×4×⅜; 2-STIFF. b 4×3½×5/16}
TOP ∠ - 4×3×5/16 - ta
AMES DIAL
3⅝"

FIG. 15

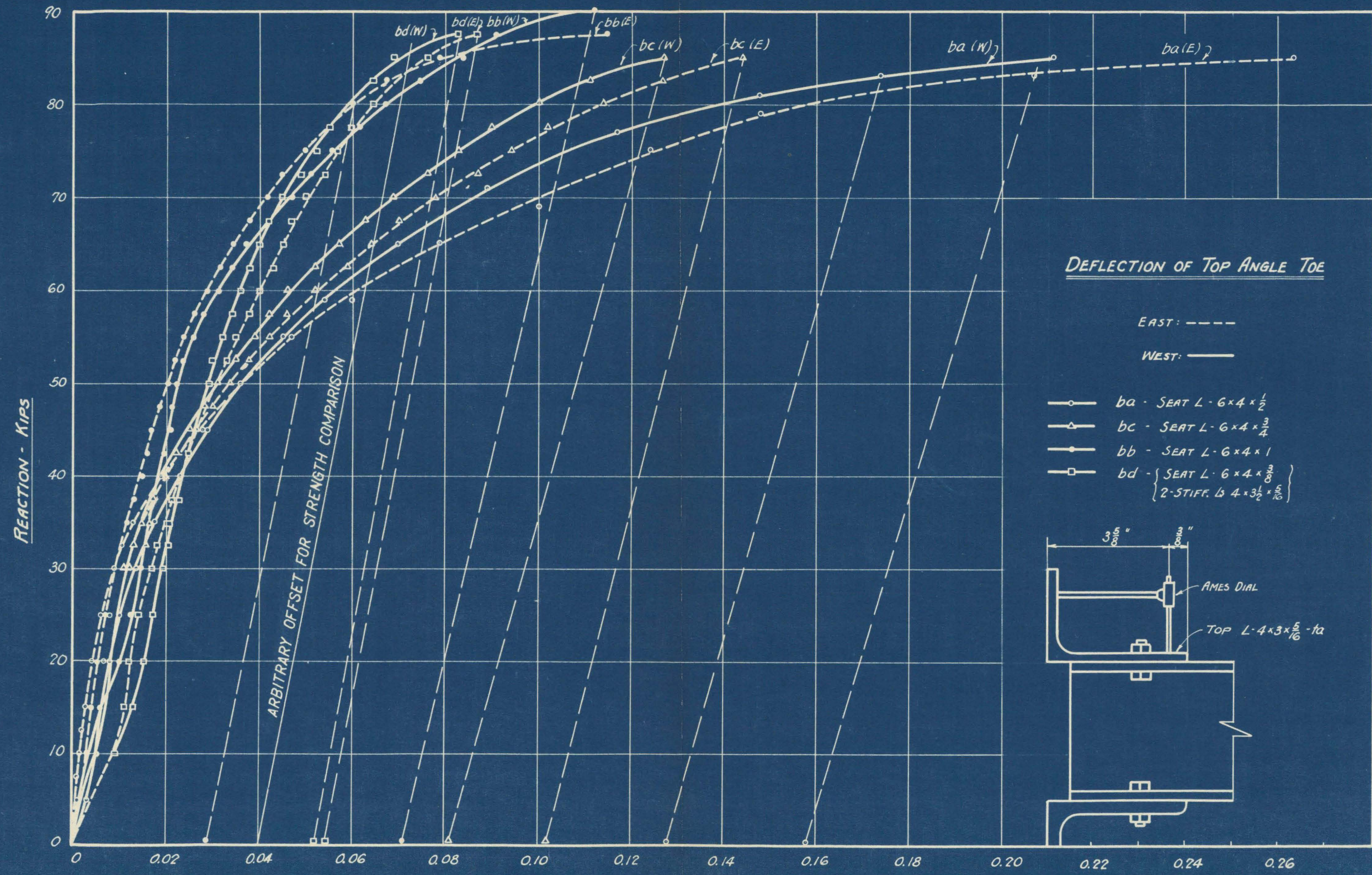


FIG. 16 DEFLECTION OF TOP ANGLE TOE - INCHES

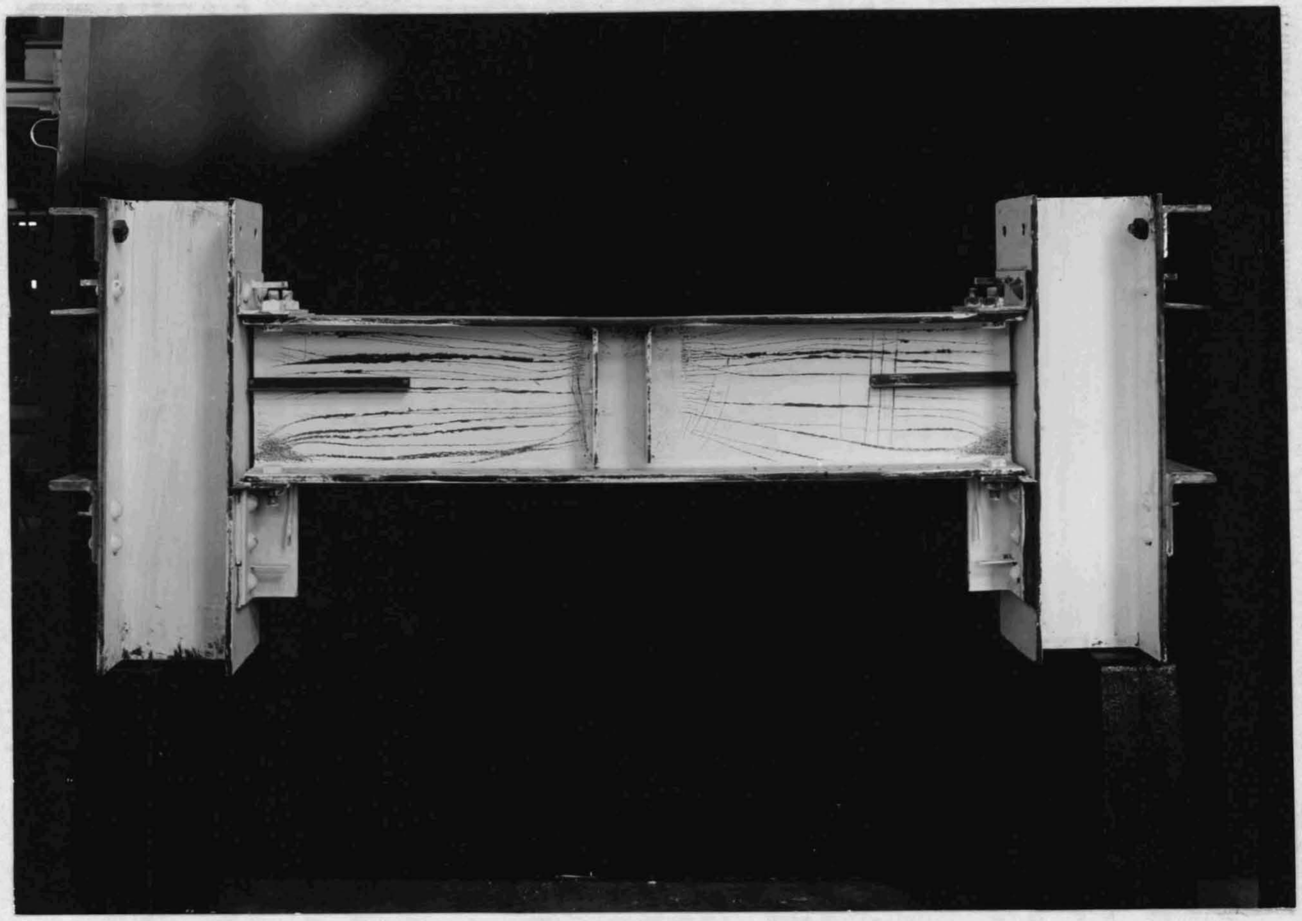


Fig. 17 - Test Specimen bd After Test

APPENDIX A

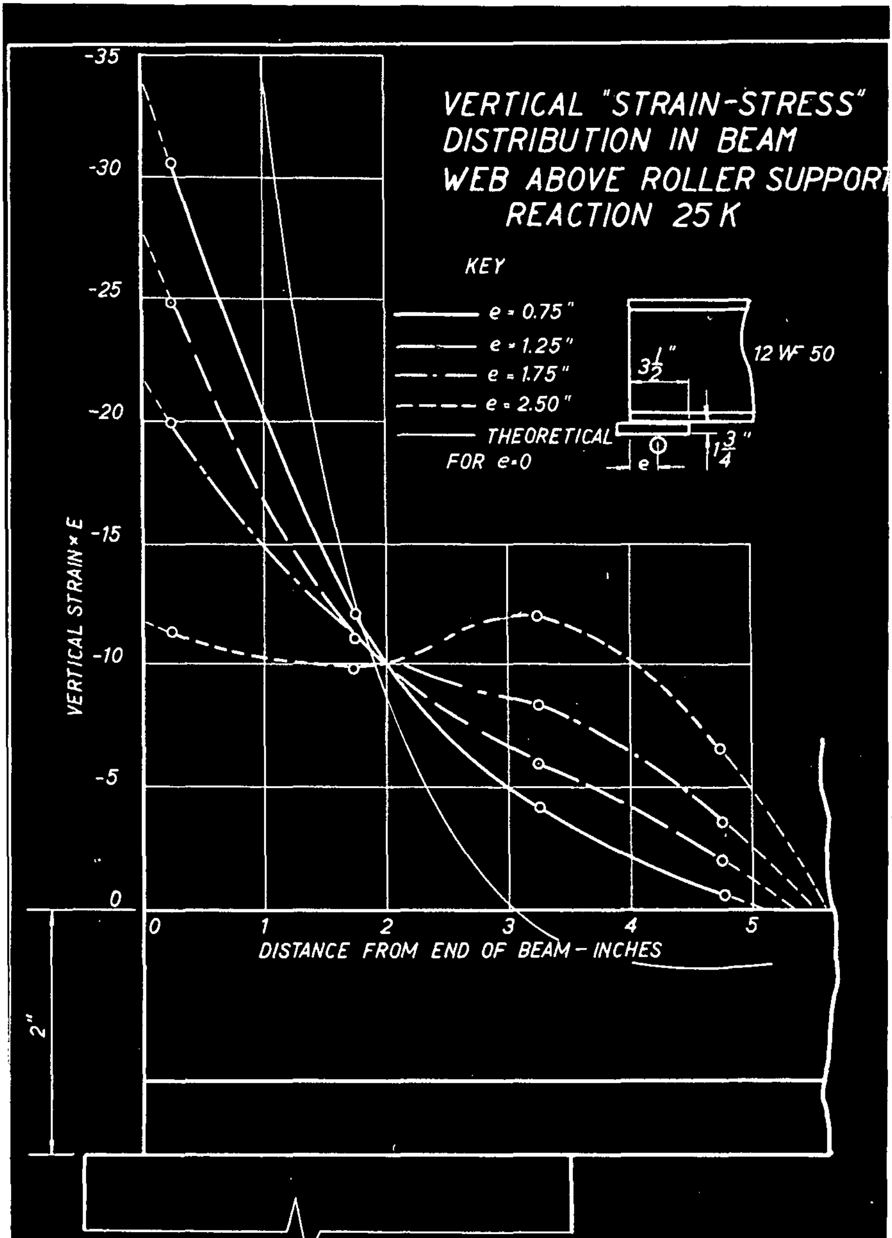
The results of preliminary tests made by Mr. R. A. Hechtman are shown on the accompanying drawing. The vertical strain distribution in the web along a line two inches from the bottom of the beam was determined for four different roller positions as shown. The test details and technique were quite similar to that used in the elastic range for the main test program. Huggenberger tensometers were used to measure strains over a one-inch gage length.

Although the test reaction was 25 kips in each case the stress areas shown account for uniformly low reactions of 20.2, 19.3, 20.1, and 20.4 kips for e = 0.75, 1.25, 1.75, and 2.50 in. respectively. This discrepancy may be due in part to two causes:

(1) Horizontal strains were not measured, hence $E \epsilon_y$ rather than σ_y is computed.

(2) Part of the load is carried in shear by web material below the gage line.

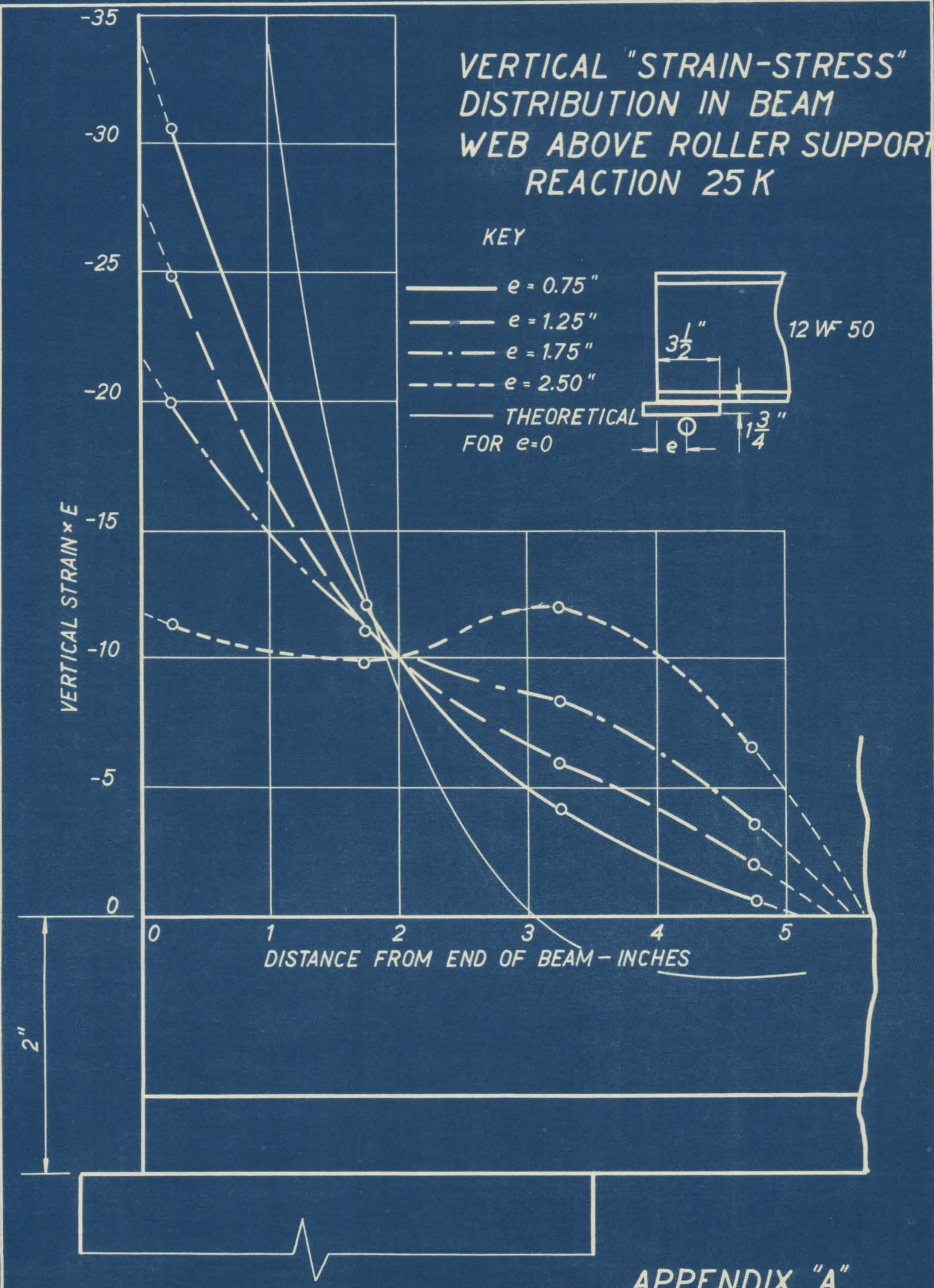
The diagram is of interest in showing the relative variation in stress distribution for different positions of the roller support.



VERTICAL "STRAIN-STRESS" DISTRIBUTION IN BEAM WEB ABOVE ROLLER SUPPORT
REACTION 25 K
KEY
e = 0.75"
e = 1.25"
e = 1.75"
e = 2.50"
THEORETICAL FOR e=0
12 WF 50
3 1/2"
1 3/4"
e
VERTICAL STRAIN × E
-35
-30
-25
-20
-15
-10
-5
0
0
1
2
3
4
5
DISTANCE FROM END OF BEAM - INCHES
2"

APPENDIX "A"

7/82/23-6A


VERTICAL "STRAIN-STRESS" DISTRIBUTION IN BEAM WEB ABOVE ROLLER SUPPORT
REACTION 25 K
KEY
e = 0.75"
e = 1.25"
e = 1.75"
e = 2.50"
THEORETICAL FOR e=0
12 WF 50
3 1/2"
1 3/4"
e
VERTICAL STRAIN × E
-35
-30
-25
-20
-15
-10
-5
0
0
1
2
3
4
5
DISTANCE FROM END OF BEAM - INCHES
2"

APPENDIX "A"

APPENDIX B

Mr. F. H. Frankland
Director of Engineering, A.I.S.C.
101 Park Avenue, New York City

July 19, 1941.

Shearing Stresses in Steel Beams

Dear Mr. Frankland:

As a side light in connection with the recent tests of short beams supported by seat angles at each end certain facts have been noted regarding the failure of these beams by horizontal shear.

At a total center load of 170 kips, or 85 kips shear, each beam had yielded rather generally throughout the beam web area. The average shear stress at failure, based on gross web area, depth "d" times web thickness "t", gives an average shear stress at failure of 19.1 kips per sq in. This gives a factor of safety with respect to the 13 kips per sq in. which is allowed of only 1.47. The web material had an average yield strength of about 40 kips per sq in., therefore the adjusted factor of safety would be only 1.21.

Several years ago the writer wrote a brief note on this subject for the "Civil Engineering" magazine of April 1938, p. 273, from which the following is quoted:

> "In a short beam centrally loaded, the maximum shear stress has a more definite relation to the structural failure of the beam than does the maximum direct stress in a long beam centrally loaded. This is because the shear stress critically affects a proportionately greater portion of the total beam than do the (maximum bending stresses. Hence it is quite as important in certain cases to design for maximum shear stress as for maximum direct stress."

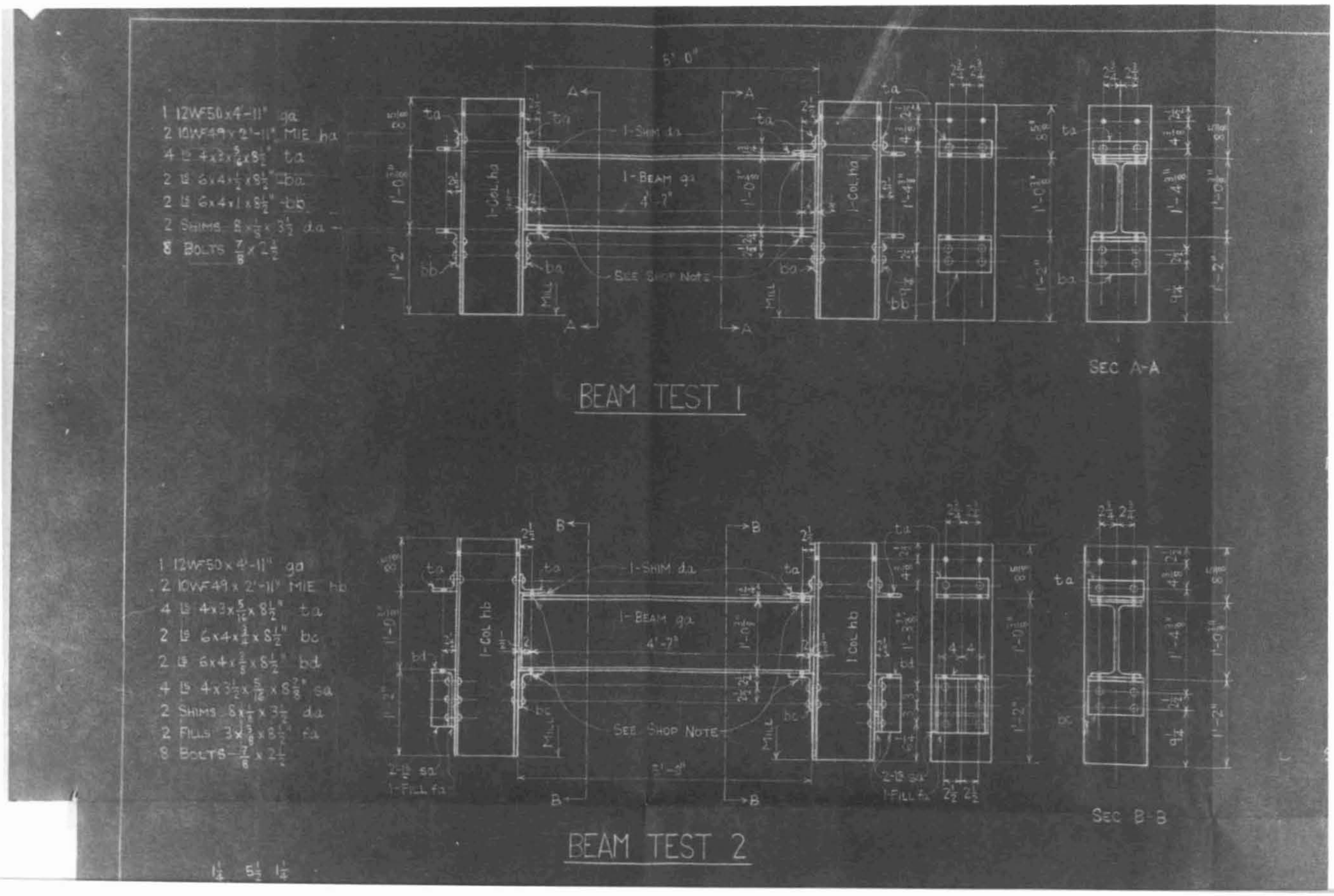
The writer than gave a simple formula for approximating the maximum shear stress within about one per cent (discounting concentration effects). Thisfformula is as follows:

$$v_{max} = \frac{V}{tI}\left[\left(\frac{A - A_w}{4}\right)(d - f) + \frac{A_w d}{8}\right]$$

I = moment of inertia; A = gross area;
A_w = web area; d = depth; f = flange thickness

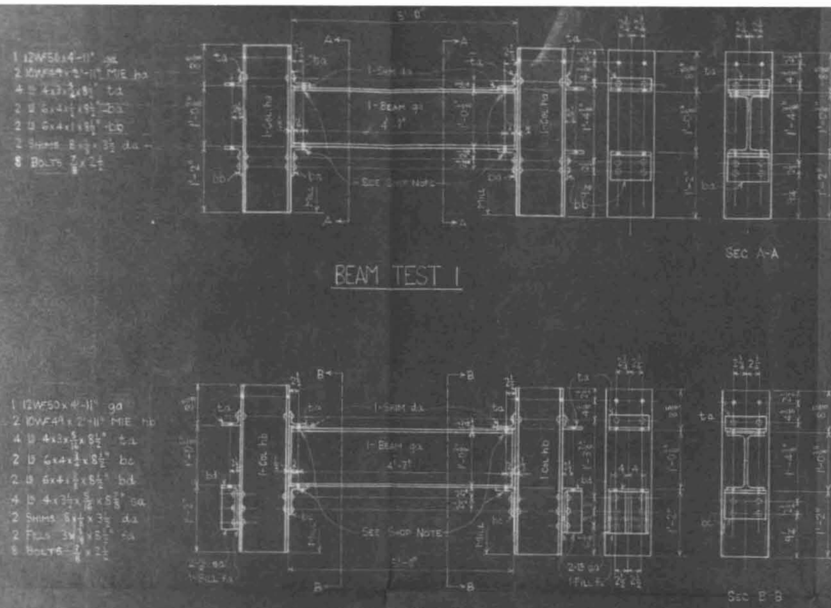
According to the above formula the maximum shear stress at failure was 21 kips per sq in., which would give a factor of safety of 1.61 if the 13 kips per sq in were maximum rather than average allowable shear stress. This adjusts to 1.33 for the factor of safety for minimum yield-point strength. Even this is low, but the margin of difference between it and 1.21 is considerable.

Very truly yours
(Signed) Bruce Johnston



	REQUIRED	
1	Beam Test	1
1	Beam Test	2
2	Beams	GA
4	Shims	DA
24	Bolts ⅞" x 2½"	
	Coupons or Plain Material To Be Furnished	
1	12WF50 x 2'-8"	
1	10WF49 x 1'-4"	
1	L 6x4x½x1'-4"	
1	L 6x4x¾x1'-4"	
1	L 6x4x1x1'-4"	
1	L 6x4x⅜x1'-4"	
1	L 4x3x5/16x1'-4"	
1	L 4x3½x5/16x1'-4"	

Beams Billed "GA" Are Same As "ga"
Shims Billed "DA" Are Same As "da"



Shop Note: Clip Angles Are To Be Riveted To The Column Sections Before Beams Are Placed. Beams For Beam Tests 1 And 2 Are To Be Bolted Complete With As Many Shims Under Top Angle As Necessary.

Material: All Material to Meet A.S.T.M. Standard Specifications A-7-39 for Bridge and Building Steel and to Be Free from Rust and Paint with Mill Scale Intact. All Sections of Same Size, Including Coupons, to Be Cut from Same Piece of Stock.

Coupons: Plain Material to Be Furnished as Billed.

Holes: Punch 15/16" ϕ. Open Holes Noted.

Rivets: ⅞" ϕ. All Rivets to Be Driven By Pneumatic Hammers By Methods Corresponding to Best Field Driving Practice.

Bolts: Bolt Complete ⅞ ϕ.

Paint: No Paint.

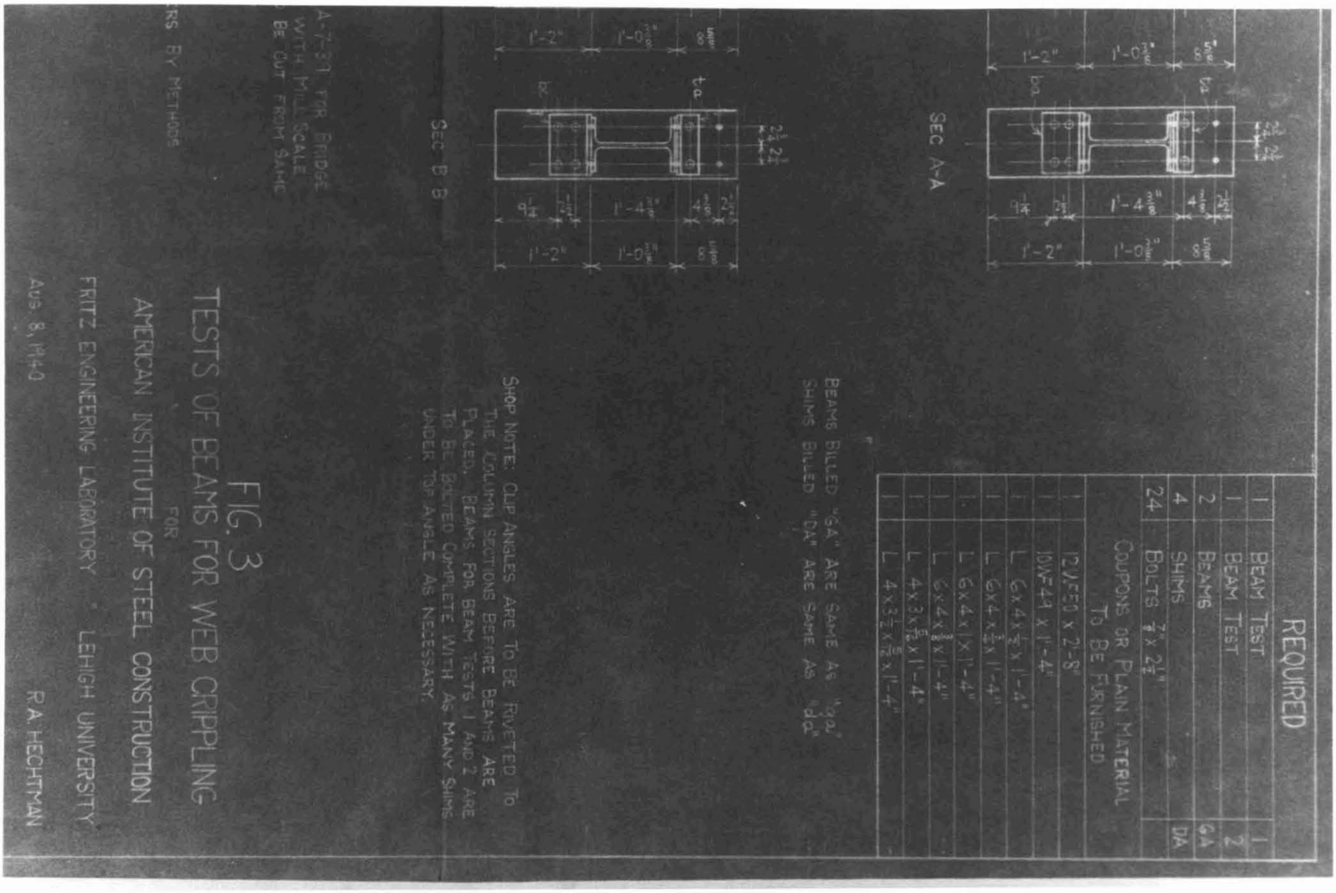
FIG. 3
TESTS OF BEAMS FOR WEB CRIPPLING
FOR
AMERICAN INSTITUTE OF STEEL CONSTRUCTION

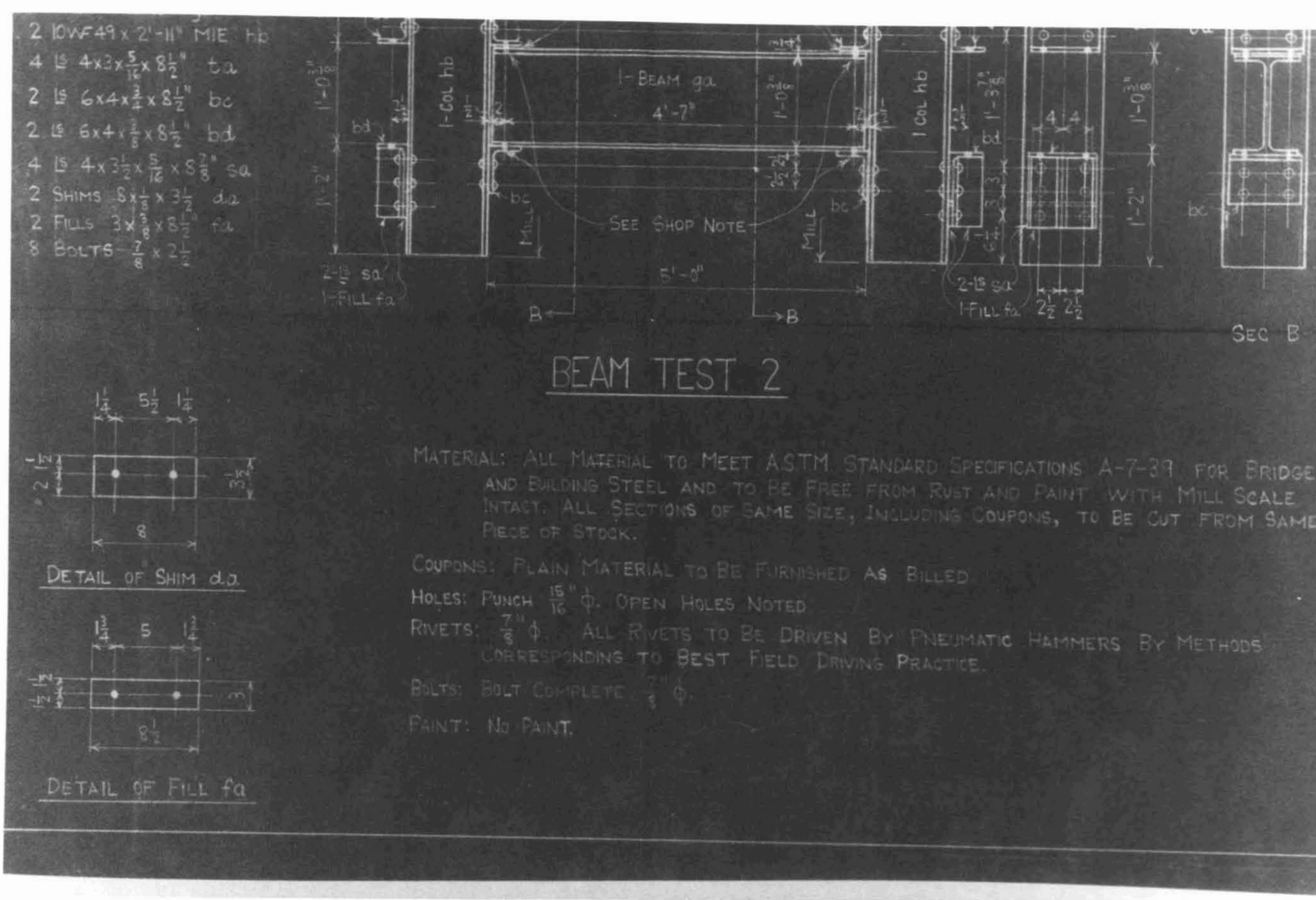
FRITZ ENGINEERING LABORATORY LEHIGH UNIVERSITY

Aug. 8, 1940 R.A. HECHTMAN

7/82/23-16A

7/82/23-13A

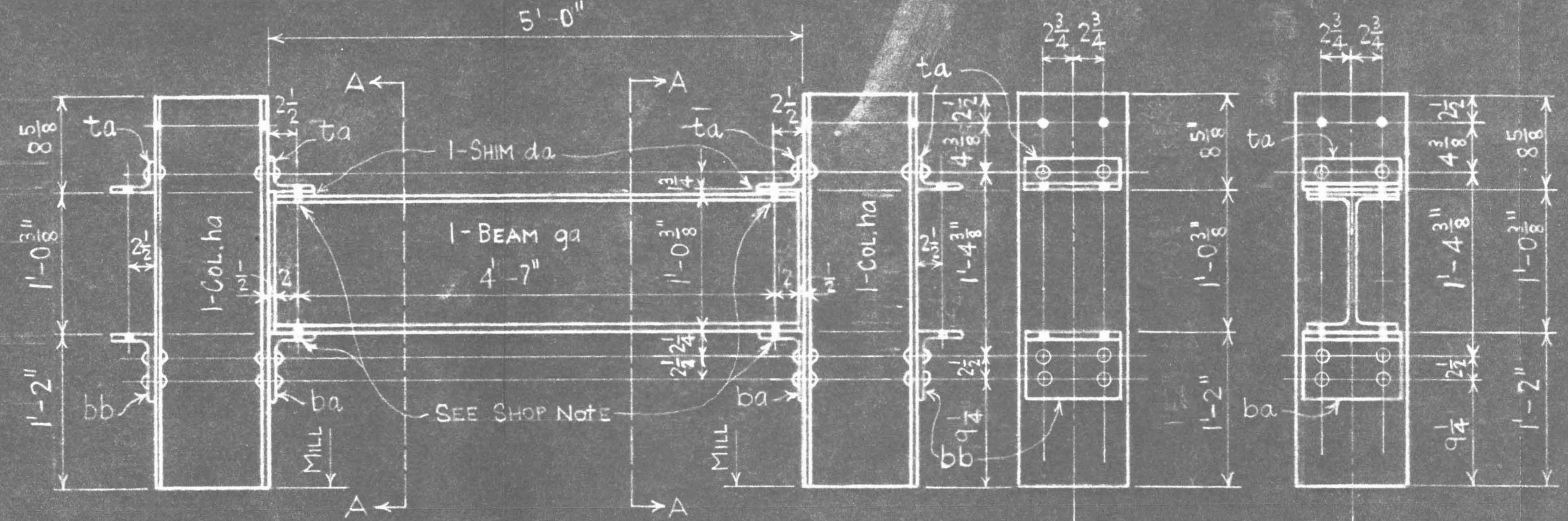

SEC A-A
SEC B-B
REQUIRED
1 BEAM TEST 1
1 BEAM TEST 2
2 BEAMS GA
4 SHIMS DA
24 BOLTS 7/8" x 2 1/4"
COUPONS OR PLAIN MATERIAL
TO BE FURNISHED
1 12VF50 x 2'-8"
1 10WF49 x 1'-4"
1 L 6x4x3/8 x 1'-4"
1 L 6x4x1/2 x 1'-4"
1 L 6x4x1 x 1'-4"
1 L 6x4x3/8 x 1'-4"
1 L 4x3x5/16 x 1'-4"
1 L 4x3 1/2 x 5/16 x 1'-4"
BEAMS BILLED "GA" ARE SAME AS "ga"
SHIMS BILLED "DA" ARE SAME AS "da"
SHOP NOTE: CLIP ANGLES ARE TO BE RIVETED TO THE COLUMN SECTIONS BEFORE BEAMS ARE PLACED. BEAMS FOR BEAM TESTS 1 AND 2 ARE TO BE BOLTED COMPLETE WITH AS MANY SHIMS UNDER TOP ANGLE AS NECESSARY.
FIG. 3
TESTS OF BEAMS FOR WEB CRIPPLING
FOR
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
FRITZ ENGINEERING LABORATORY LEHIGH UNIVERSITY
AUG. 8, 1940
R.A. HECHTMAN


2 10WF49 x 2'-11" MIE hb
4 Ls 4x3x5/16 x 8 1/2" ta
2 Ls 6x4x3/4 x 8 1/2" bc
2 Ls 6x4x3/8 x 8 1/2" bd
4 Ls 4x3 1/2 x 5/16 x 8 7/8" sa
2 SHIMS 8x1/8 x 3 1/2 da
2 FILLS 3x3/8 x 8 1/2 fa
8 BOLTS 7/8 x 2 1/2
1-BEAM ga
4'-7"
1 COL hb
SEE SHOP NOTE
5'-0"
2-Ls sa
1-FILL fa
SEC B-B
BEAM TEST 2
MATERIAL: ALL MATERIAL TO MEET A.S.T.M. STANDARD SPECIFICATIONS A-7-39 FOR BRIDGE AND BUILDING STEEL AND TO BE FREE FROM RUST AND PAINT WITH MILL SCALE INTACT. ALL SECTIONS OF SAME SIZE, INCLUDING COUPONS, TO BE CUT FROM SAME PIECE OF STOCK.
COUPONS: PLAIN MATERIAL TO BE FURNISHED AS BILLED.
HOLES: PUNCH 15/16" φ. OPEN HOLES NOTED.
RIVETS: 7/8" φ. ALL RIVETS TO BE DRIVEN BY PNEUMATIC HAMMERS BY METHODS CORRESPONDING TO BEST FIELD DRIVING PRACTICE.
BOLTS: BOLT COMPLETE 7/8" φ.
PAINT: NO PAINT.
DETAIL OF SHIM da
DETAIL OF FILL fa

7/82/23-14A

7/82/23-15A

1 12WF50 x 4'-11" ga
2 10WF49 x 2'-11" MIE ha
4 Ls 4x3x$\frac{5}{16}$x8$\frac{1}{2}$" ta
2 Ls 6x4x$\frac{1}{2}$x8$\frac{1}{2}$" ba
2 Ls 6x4x1x8$\frac{1}{2}$" bb
2 Shims 8x$\frac{1}{8}$x3$\frac{1}{2}$ da
8 Bolts $\frac{7}{8}$x2$\frac{1}{2}$

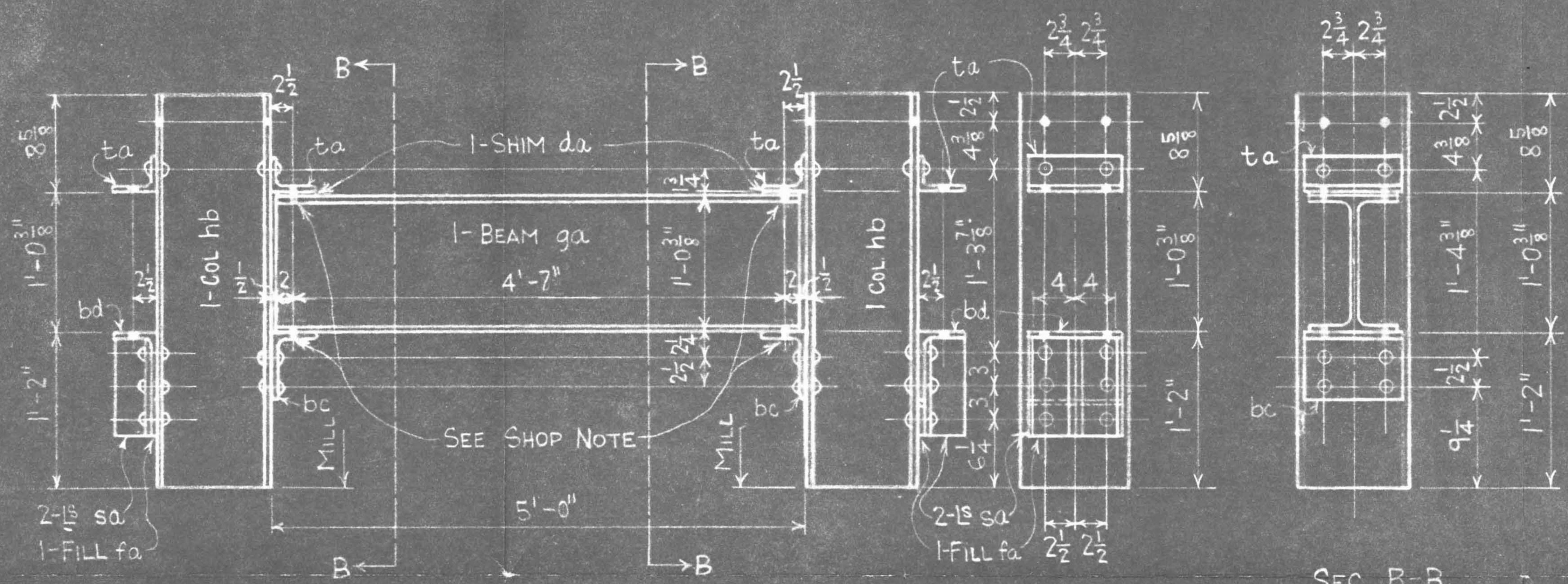


BEAM TEST 1

	REQUIRED	
1	Beam Test	1
1	Beam Test	2
2	Beams	GA
4	Shims	DA
24	Bolts $\frac{7}{8}$" x 2$\frac{1}{2}$"	
	Coupons or Plain Material To Be Furnished	
1	12WF50 x 2'-8"	
1	10WF49 x 1'-4"	
1	L 6x4x$\frac{1}{2}$ x 1'-4"	
1	L 6x4x$\frac{3}{4}$ x 1'-4"	
1	L 6x4x1x1'-4"	
1	L 6x4x$\frac{3}{8}$ x 1'-4"	
1	L 4x3x$\frac{5}{16}$ x 1'-4"	
1	L 4x3$\frac{1}{2}$x$\frac{5}{16}$ x 1'-4"	

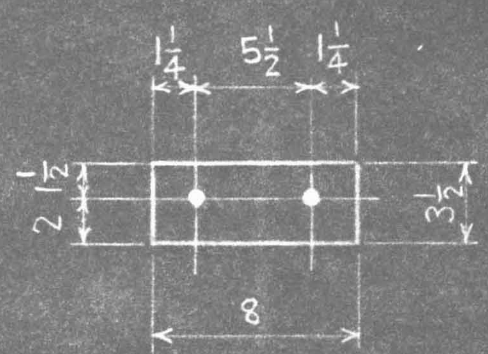
Beams Billed "GA" Are Same As "ga"
Shims Billed "DA" Are Same As "da"

1 12WF50 x 4'-11" ga
2 10WF49 x 2'-11" MIE hb
4 Ls 4x3x$\frac{5}{16}$x8$\frac{1}{2}$" ta
2 Ls 6x4x$\frac{3}{4}$x8$\frac{1}{2}$" bc
2 Ls 6x4x$\frac{3}{8}$x8$\frac{1}{2}$" bd
4 Ls 4x3$\frac{1}{2}$x$\frac{5}{16}$x8$\frac{7}{8}$" sa
2 Shims 8x$\frac{1}{8}$x3$\frac{1}{2}$ da
2 Fills 3x$\frac{3}{8}$x8$\frac{1}{2}$" fa
8 Bolts $\frac{7}{8}$x2$\frac{1}{2}$



BEAM TEST 2

Shop Note: Clip Angles Are To Be Riveted To The Column Sections Before Beams Are Placed. Beams For Beam Tests 1 And 2 Are To Be Bolted Complete With As Many Shims Under Top Angle As Necessary.



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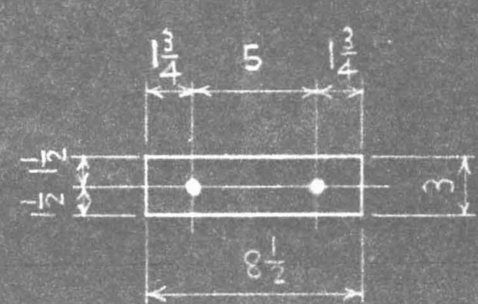

Detail of Fill fa

Material: All Material to Meet A.S.T.M. Standard Specifications A-7-39 for Bridge and Building Steel and to Be Free from Rust and Paint with Mill Scale Intact. All Sections of Same Size, Including Coupons, to Be Cut from Same Piece of Stock.

Coupons: Plain Material to Be Furnished As Billed.

Holes: Punch $\frac{15}{16}$" ϕ. Open Holes Noted.

Rivets: $\frac{7}{8}$" ϕ. All Rivets to Be Driven By Pneumatic Hammers By Methods Corresponding to Best Field Driving Practice.

Bolts: Bolt Complete $\frac{7}{8}$" ϕ.

Paint: No Paint.

FIG. 3
TESTS OF BEAMS FOR WEB CRIPPLING
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AUG. 8, 1940 R.A. HECHTMAN

4 negatives for this blueprint
top left 7/82/23-13A
2 bottom left 7/82/23-16A - 7/82/23-15A
right side 7/82/23-14A